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# FOR
# TESTING MATERIALS

ORGANIZED IN 1898
INCORPORATED IN 1902

# PROCEEDINGS
## OF THE
# THIRTY-FOURTH ANNUAL MEETING

**Held at Chicago, Illinois**
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# American Society for Testing Materials

ORGANIZED IN 1898

INCORPORATED IN 1902

## PROCEEDINGS, PART II

The Society is not responsible, as a body, for the statements and opinions advanced in this publication.

## SUMMARY OF PROCEEDINGS OF THE PITTSBURGH REGIONAL MEETING—SYMPOSIUM ON WELDING

The second regional meeting of the Society was held in Pittsburgh at the Hotel William Penn on March 18, 1931, under the auspices of the Pittsburgh District Committee, in conjunction with the Spring Group Meeting of A.S.T.M. committees. The meeting comprised a technical program, a Symposium on Welding, to which both morning and afternoon sessions were devoted, followed by a dinner in connection with which there was a demonstration on the analysis and synthesis of sound arranged through the courtesy of the Research Laboratories of the Westinghouse Electric and Manufacturing Co. President K. G. Mackenzie and the Secretary-Treasurer of the Society were the speakers at the dinner. There were over 500 in attendance at the meeting.

The Welding Symposium was arranged with the cooperation of the Pittsburgh Section of the American Welding Society and consisted of twelve papers as follows:

*Morning Session*

"General Survey of Welding Processes," by F. T. Llewellyn, United States Steel Corporation.

"Welding Processes Applicable to Aluminum," by W. M. Dunlap, Metallurgist, Metallurgical Division, Research Laboratories, Aluminum Company of America.

"The Quality of Materials for Fusion Welding," by C. R. Texter and F. N. Speller, Department of Metallurgy and Research, National Tube Co.

"Modern Applications of Arc Welding," by A. M. Candy, Welding Engineer, Westinghouse Electric and Manufacturing Co.

"Recent Developments in Gas Welding and Cutting," by E. J. W. Egger, Development Engineer, Linde Air Products Co.

### *Afternoon Session*

"Stethoscopic Examination of Welded Products," by J. R. Dawson, Metallurgical Engineer, Union Carbide and Carbon Research Laboratories, Inc.

"Inspection of Welding Made by the Watertown Arsenal," by G. F. Jenks, Lieutenant Colonel, Ordnance Dept., U. S. Army, Watertown Arsenal.

"Gamma-Ray Testing of Welds," by G. E. Doan, Associate Professor of Physical Metallurgy, Lehigh University.

"Magnetic Methods of Testing Butt Welds," by T. R. Watts, Research Engineer, Research Laboratories, Westinghouse Electric and Manufacturing Co.

"Fatigue and Impact Testing of Welded Products," by T. M. Jasper, Director of Research, A. O. Smith Corporation.

"Welding Inspection," by R. Kraus, Supervisor of Welding Inspection, Westinghouse Electric and Manufacturing Co.

The chairman of the sessions was Mr. G. H. Danforth, Past-President, Engineering Society of Western Pennsylvania. The papers elicited a considerable amount of discussion which contributed greatly to the information contained in the papers. The papers, complete with discussion, have been published by the Society as a separate volume entitled, "Symposium on Welding."

# SUMMARY OF PROCEEDINGS OF THE SYMPOSIUM ON EFFECT OF TEMPERATURE ON THE PROPERTIES OF METALS

The Symposium on Effect of Temperature on the Properties of Metals was held at a joint meeting of the American Society of Mechanical Engineers and the American Society for Testing Materials in conjunction with the annual meeting of the latter Society at The Stevens, Chicago, June 23, 1931. Two sessions were held, one in the afternoon and the other in the evening. Mr. K. G. Mackenzie, President of the A.S.T.M., presided in opening both sessions, with Messrs. A. D. Bailey and H. W. Gillett, chairman and secretary, respectively, of the first session, and Messrs. F. M. Becket and H. W. Gillett, chairman and secretary, respectively, of the second session. The symposium was introduced by Mr. W. L. Abbott, Past-President of the A.S.M.E.

The program had been developed by the Joint A.S.M.E.-A.S.T.M. Research Committee on the Effect of Temperature on the Properties of Metals and covered the two major phases of the subject, the first, Engineering Trends and Requirements for Metals at High and Low Temperatures, comprising 11 papers, and the second, Properties of Available Metals for High- and Low-Temperature Service, comprising 16 papers. The complete list of papers follows:

### *Afternoon Session*

### I. ENGINEERING TRENDS AND REQUIREMENTS FOR METALS AT HIGH AND LOW TEMPERATURES

"Trends in Engineering Requirements for Metals for the Power Plant Industry —Steam Boiler and Superheater Division," by H. J. Kerr.

"Steam Turbine Materials for High Temperatures," by R. C. Allen.

"Metallurgical Requirements for High-Temperature Steam Piping," by F. W. Martin.

"Needs of the Oil Industry for Metals at High Temperatures," by E. S. Dixon.

"Corrosion and Other Problems in the Use of Metals at High Temperatures in the Chemical Industries," by F. H. Rhodes.

"Engineering Requirements and Trends for Metals in the Ferrous Metal Industries," by J. C. Woodson.

"Engineering Requirements and Trends for Metals in the Non-Ferrous Roasting, Smelting and Refining Industry," by R. E. Brown.

"Engineering Requirements in the Automotive Industry for Metals Operating at High Temperatures," by A. L. Boegehold and J. B Johnson.

"Use of Metals at Elevated Temperatures in the Ceramic Industry," by C. E. Williams.

"The Trend of Progress in Great Britain on the Engineering Use of Metals at Elevated Temperatures," by R. W. Bailey, J. H. S. Dickenson, N. P. Inglis and J. L. Pearson.

"The Mechanical Properties of Metals at Elevated Temperatures," by Pierre Chevenard.

*Evening Session*

## II. PROPERTIES OF AVAILABLE METALS FOR HIGH- AND LOW-TEMPERATURE SERVICE

"Zinc Alloys for High- and Low-Temperature Service," by H. A. Anderson.
"The Mechanical Properties of Aluminum and Magnesium Alloys at Elevated Temperatures," by R. L. Templin and D. A. Paul.
"Properties of Bearing Metals at Normal and Elevated Temperatures," by E. R. Darby.
"Properties of Copper and Some of Its Important Industrial Alloys at Elevated Temperatures," by W. B. Price.
"High-Temperature Creep Properties of Wrought Carbon and Low-Alloy Steels," by A. P. Spooner and F. B. Foley.
"The Performances of Cast-Carbon and Low-Alloy Steels in High- and Low-Temperature Service," by R. A. Bull.
"Effect of Elevated Temperatures on Certain Mechanical Properties of Gray Cast Iron and Malleable Iron," by J. W. Bolton and Hyman Bornstein.
"The Wrought Austenitic Alloys," by R. H. Aborn and E. C. Bain.
"The Effect of Temperature on Some Properties of Iron-Chromium-Nickel Alloys," by N. B. Pilling and Robert Worthington.
"Nickel and Nickel Alloys Other than the Nickel-Chromium-Iron Group," by C. A. Crawford and Robert Worthington.
"High-Chromium Steels for Extreme Service Conditions," by C. E. MacQuigg.
"Properties of the Rare Metals for High-Temperature Service," by W. H. Swanger.
"Effect of Temperature on the Properties of Nitrided Alloys," by O. E. Harder.
"Effect of Low Temperatures on Metals and Alloys," by H. W. Russell.
"Thermal Expansion of Metals," by N. L. Mochel.
"Note on Applications of Data on the Thermal Conductivity of Metals," by M. S. Van Dusen.

All of the papers, including a "Supplementary Bibliography on Effect of Temperature on Properties of Metals" by Lois F. McCombs, had been preprinted and distributed in advance of the meeting. In order to afford adequate time for the presentation of discussion, the papers were not presented at the meeting but were summarized by two abstractors. The first group of papers, on Engineering Trends and Requirements for Metals at High and Low Temperatures, was presented in abstract by Mr. L. W. Spring and the second group of papers, on Properties of Available Metals for High- and Low-Temperature Service, was presented in abstract by Mr. H. J. French. The papers were then discussed individually, precedence being given to written discussion, of which 70 had been submitted.

The papers, complete with discussion, have been assembled in a separate volume entitled, "Symposium on Effect of Temperature on the Properties of Metals," published jointly by the two Societies.

# THE PHENOMENON OF SLIP IN PLASTIC MATERIALS

## EDGAR MARBURG LECTURE[1]

By A. NADAI[2]

## SYNOPSIS

While in former times the engineer was perhaps satisfied and it was sufficient for him to base his objects of design and his computations on the theory of elasticity, which enabled him to find the working stresses in machine parts under the assumption that these latter are stressed purely elastically, recent improvements in design and in manufacturing processes are due to a better knowledge of the permanent or plastic deformations of the ductile metals. Plasticity has been recognized as one of the general properties of solid matter, which can be utilized in a number of manufacturing processes. In metals, in minerals and rocks, most interesting evidences of regular structural changes have been found after exposing these materials to severe plastic strains and deformations.

In the description of the states of equilibrium in loose granular materials and of the plastic state under which metals flow in the testing laboratory, and solid rocks have apparently been severely deformed in great depths on a large scale in nature, an attempt has been made to explain certain frequently observed phenomena connected intimately with the limiting conditions of equilibria and to treat these latter from common points of view in idealizing certain properties of solid matter.

On metal or rock specimens subjected to severe stressing, faint traces of locally concentrated deformations in thin layers have been observed. In these thin layers visible on the surface of a test specimen, frequently a pattern with great regularity is disclosed. The strange laws under which they appear have recently attracted the mathematicians. These regular markings might offer useful means to study the distributions of stress under which they were produced. To these thin plastic layers, quite analogous phenomena seem to correspond in geology on a large scale. In a greater number of examples, taken from practical applications in engineering design and from the branch science of structural geology, the laws of the formation of the slip lines and of the plastic layers have been studied.

Through the courtesy of the Research Laboratories of the Westinghouse Electric and Manufacturing Co. in East Pittsburgh, a new optical bench has been designed by the speaker, which serves to demonstrate and to study the formation of the plastic slip layers in test specimens, with a high degree of accuracy.

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[1] Read on June 24, 1931, before the Annual Meeting of the American Society for Testing Materials, Chicago, Ill.

[2] Research Engineer, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa. Formerly Professor at the University of Goettingen.

It is an indication of the farsighted aims which dominate the members of the American Society for Testing Materials that there is kept alive a perpetual expression of recognition for the life work of the worthy founder and pioneer of the Society. To honor the memory of Edgar Marburg, who devoted a considerable portion of a lifetime of effort to promoting knowledge of the properties of engineering materials, it would seem fitting at this time to cast a brief retrospective glance at some developments in one or two secluded provinces of the wide territory embracing the present knowledge of the mechanical properties of engineering materials. It is a high honor to have been called to present this commemorative address in behalf of the memory of the distinguished founder of the Society, for which the speaker here wishes to express his sincere gratitude and thanks to the American Society for Testing Materials. As a gesture of recognition, some facts in a specific field of investigation might be briefly enumerated.

In our present age of specialization, it might seem in order to try to describe a group of various phenomena from a common point of view. Among our increasingly diverging interests in the present developments in science or engineering, it would appear a timely and fascinating task to attempt to summarize and correlate a group of empirically gathered facts. In witnessing developments which have taken place during the last few decades, the continuous trend towards the gradual formation of new branches in science or in engineering knowledge stands out prominently. The process repeats itself from time to time, although the reasons for the formation of new branches in science or engineering may differ. In some cases the urgent practical needs of our daily industrial life, in other instances occasional coincidences, favorable to certain developments, or circumstances creating new needs after a period of rapid changes are the immediate causes of new developments. Ultimately perhaps the persistent faith of a worker in a single thought, in the everlasting order governing the happenings in nature, and his devotion to a specific object of his imagination are the prevalent driving forces, which break the grounds for new developments. Mostly these edifices grow slowly, after stone by stone has been laid down by innumerable single efforts until a more comprehensive whole becomes distinctly perceptible.

In such a way, for example, the efforts of the physicochemists and metallurgists have finally created the branch sciences of metallography and of metallurgy, by basing their investigations of the physical or chemical properties of the metallic elements, of their alloys, and of the systems of two or more components on what might be called thermal analysis and also thermochemical principles.

In recent years on the other hand, the physicist has been attracted by the fascinating properties of the single crystals of the metallic elements, and the chemist and the mechanical engineer have joined him in his efforts to describe the modes of deformation of these specimens of metallic materials, which first were the products of occasionally mistreated metals, but later on were found perhaps to represent certain properties of the metallic elements to great perfection. By systematically investigating the physical and the mechanical properties of the single metallic crystals, that is, of the metals in the most perfect state of aggregation of their elementary particles, valuable information could be obtained relative to the mechanical properties, strength, and plastic deformations of the metals, and these investigations have thereby contributed more to the knowledge of the solid state of matter and of the strength of solids than could have been obtained using only the early methods of testing materials.

Attention might be called to a further group of questions and a group of materials with properties in sharp contrast to those mentioned above.

The loose, earthy masses and soils are materials of great interest to the engineer, not so much because they may be applied for certain structural purposes in dams, embankments, and roads, but more because of their importance in connection with the foundations of buildings and engineering edifices. Relative to the mechanical properties of the loose earthy materials as dependent on their water or moisture content, valuable data have been gathered during the past few decades and an adequate statement might perhaps be to say that a new science of foundations and of the mechanical properties of wet earthy or colloidal masses which play a part in the economy of large engineering enterprises, such as river improvements, excavations for large canals, and foundations on soft earth, is in the process of formation.

With regard to the equilibrium conditions of the parts along the walls of a fresh cut or excavation in wet earthy masses, countless observations have shown that the walls cannot stand up unless their inclination to the horizontal is less steep than a certain limiting angle. The water content within the adjoining material of retaining walls and any cause for a disturbance in the invisible slow processes of diffusion of moisture through wet clay or in the more readily detectable flow of the ground water through the pores of loose earthy masses will affect the equilibrium condition and have an influence on the steepness with which the walls may be cut without danger of sliding.

## MODELS OF VARIOUS STATES OF MATTER

In mechanics it is useful to ascribe to materials certain ideal properties. In attributing to them certain definite, simple properties, it is possible in a number of cases of practical interest to predict quantitatively the velocities or the pressures at internal points of a moving mass, the states of internal strain or stress in material bodies or to describe and predict more exactly the mechanical conditions under which, in a given case, engineering structures rupture or permanent deformations start to develop. It is always useful, when discussing the phenomena from a given point of view, to ascertain that these definitions may contain strong idealizations of the true conditions and have definite limits in their applications.

A mechanical characteristic of a perfect fluid is the easy mobility of the small elementary parts thereof, which may be expressed by stating that the pressure at any internal section in a fluid at rest is always directed perpendicularly to this section. On the other hand, the elements of a solid may carry, in addition to normal forces in internal sections, forces acting in the plane of the cross-section in a tangential direction, which, when expressed per unit of area, are termed shearing stresses. These shearing stresses may reach considerable magnitude in solids. The so-called viscous fluids occupy a position, with respect to behavior, midway between solids and liquids. At rest they behave similarly to perfect fluids. During motion, however—even during a scarcely perceptible slow change of their shape in their interior—tangential forces are produced as one infinitely thin layer slides over the neighboring one. These forces or shearing stresses increase as the fluid velocity changes along a line perpendicular to the direction of motion, and the simplest expression for this property might be that the shearing stress acting between moving parallel layers gliding over each other is proportional to the rate of the velocity in the direction perpendicular to the surface of the layer.

The so-called amorphous solids (such as glass) when loaded for a long time and exposed to sufficiently high temperatures seem to behave in a similar manner as the viscous fluids. The more solid polycrystalline materials such as the stronger metals, however, at least at comparatively low temperatures, have another characteristic property. If certain limits for the stresses are not reached they behave as purely elastic materials; that is, the small deformations which they undergo when a load is applied are reversible and disappear if the load is again taken off. However, after exceeding certain limits of stress, they start to deform slowly by permanent amounts without an appreciable increase of stress. There, again, by certain idealizations, it may be possible to determine the limiting states of stress, under which certain of these solids start to deform permanently and become plastic; also to describe the states of stress while they further so deform. It is possible to define what might be called *a perfectly plastic material* and to add now a new term for a class of material bodies to those already mentioned, if the fact of the existence of a so-called "limit of plasticity" is adequately expressed together with the kinematic laws, by means of which the state of strain is confined to purely plastic and to small deformations. This definition will imply, as we see, a condition expressing the fact that certain solids—such as mild steel for example—have a well-defined limit of yielding, under which they start to deform plastically. Hence, this condition will depend mainly on the components determining *the state of stress* in the material. Besides this "condition of plasticity," certain other facts will have to be expressed indicating how the state of strain is affected by that of stress.

A further ideal group of solids, which might occupy a position midway between the perfectly plastic materials and rigid bodies, is that of the loose granular materials. The conditions of the equilibrium in such materials will be considered below. They have in some respect great similarity to those under which a perfectly plastic material yields.

The great variation in the mechanical properties of our engineering materials seems to indicate that it will probably not be possible to describe the phenomena of plastic flow using only one or two mechanical models of a solid material. The metallic materials, the rocks and minerals differ much in their modes of deforming permanently, and temperature furthermore changes the conditions so much that it may prove useful in the description of the phenomena of plastic flow to distinguish a number of distinct idealized cases of plastic flow, such as the one which was mentioned above.

In order to determine the resistance of solids with respect to permanent change of shape more completely, as already mentioned a number of further factors must be taken in account. Certain of these factors do not lend themselves to an analysis by means of principles of mechanics and they cannot be considered here. Finally, in connection with these questions, it might here be in order to mention briefly the most valuable methods of structural analysis[1] based on the distinction between the unordered and ordered states of the elementary particles of matter, by means of which the questions with regard to the mechanical properties have been so successfully attacked from new points of view.

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[1] Saul Dushman, "Cohesion and Atomic Structure," Edgar Marburg Lecture, *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part II, p. 7 (1929).

## SURFACES OF SLIP

The surfaces which bound a *loose perfectly dry* mass of sand or of a dry granular material when held in equilibrium under gravity above a horizontal plate of a given contour have long attracted both the engineer and the mathematician. If a plate of a given contour is buried in a horizontal position in a loose mass of a granular material (such as quartz sand) and then slowly raised vertically, a certain portion of the loose powder or sandy material will remain on the plate. Observations show that a heap bounded by natural slopes forms above the plate. With heavy sands or powders, the resulting heap is limited by one of the two surfaces of constant maximum slope which can be erected above the bordering curve of the plate. These surfaces have the property that their tangent planes at any point are always inclined at the same angle to the horizontal plane; above a rectangular plane they form a surface consisting of four planes, inclined at the same

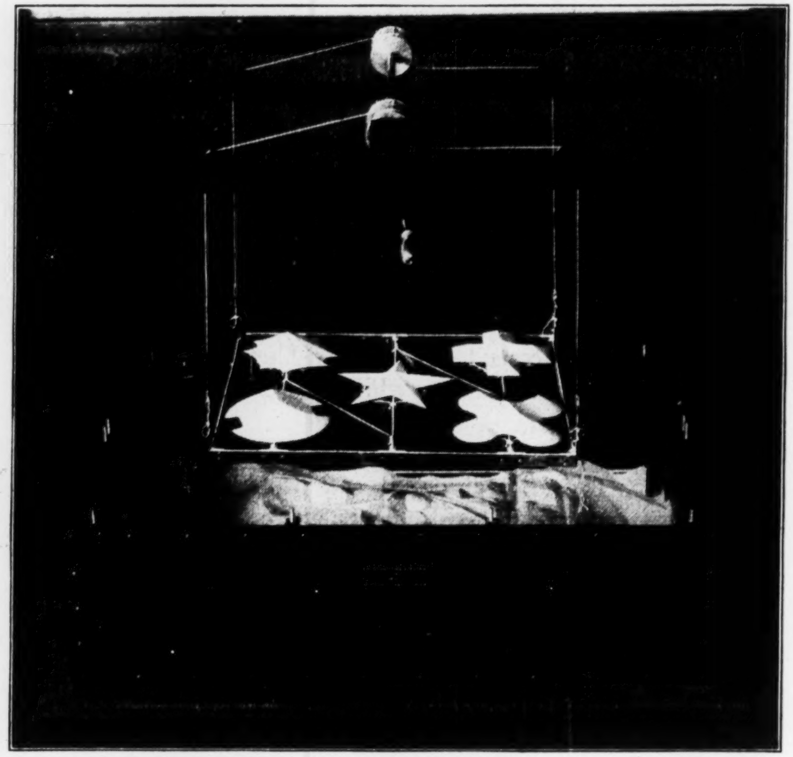


FIG. 1.—Sand-Heap Apparatus.

angles to the base which intersect in five sharp edges; above a circular plate, the surface consists of a circular cone, and so on.

It is obvious in this case, as direct observation has shown, that each linear element of the contour line of the base plate determines an element of the surface of constant slope, along which the friction acting between the grains is just sufficient to keep them in equilibrium under gravity. It is, however, perhaps less perceptible that such "surfaces of slip" which form the natural slopes of sand heaps or embankments of loose earth also might exist at any point in the interior of a loose mass.

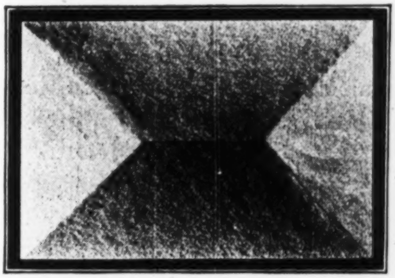


FIG. 2.—Sand Heap Showing a Surface of Constant Slope.

These surfaces of slip might be considered here for several reasons. First, as already stated, a special class of these surfaces, namely, the surfaces of constant maximum slope, determine the exact shapes of a heap of sand or of dry earth piled in equilibrium above a rigid plane base of given contour. Then they might manifest themselves in the interior of more or less loose earthy matter under the limiting states of equilibrium. An exact knowledge of these surfaces is of great practical interest to the civil engineer, since they determine the resultant forces which will act on retaining walls, under the foundations or similar elements in buildings exposed to the lateral pressure or action of a

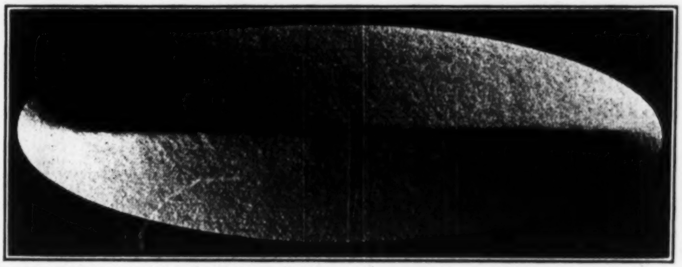


FIG. 3.—Sand Heap Showing a Surface of Constant Slope.

heavy earthy mass. Their true significance, however, is not limited by the fact that they might be used with advantage to describe more accurately the states of equilibrium in loose masses under the action of given forces. Very analogous phenomena to the formation of the surfaces of slip in loose materials have been observed in solid compact materials, such as pieces of ductile metals or of solid rocks. Under favorable conditions remarkably regular markings may be produced on the surfaces of test pieces of solid materials. These markings consist of thin layers, which appear frequently on the surface of the body as a pattern with an astonishingly regular symmetry. In these "slip layers" the material has been deformed locally more than in the adjacent material. By suitable means, these slip layers or surfaces of slip may be made visible and be studied. As these markings have proved to have a very definite relation to the fields of stress under which they have been produced, it is evident that they might be utilized for the study of the conditions under which the permanent deformations in solid materials start to develop. It has been found in fact that these thin layers of locally more deformed material have an intimate connection with the conditions under which bodies of solid (polycrystalline) matter start to slowly change their shape permanently or, in other words, under which solid bodies become plastic and start to yield.

The reasons for studying more thoroughly the laws governing the formation of these surfaces of slip in loose granular masses and of the layers of slip in compact solid bodies when brought into the plastic state might perhaps in some respect be compared with the advantages of having introduced certain ideas in the early days of physics. These latter were invented some 80 or 90 years ago for the purpose of analyzing the laws of space propagation of forces produced by electric conductors or by permanent magnets. It seems instructive to recall for a moment certain advances resulting from the introduction of conceptions so familiar to our present generation of mechanical and electrical engineers, from which new developments in mathematical physics have later originated and it might therefore seem appropriate to quote from the original texts and sources a few sentences, which in their unpretentious expressions, might still delight their readers today:

> "From my earliest experiments on the relation of electricity and magnetism, I have had to think and speak of lines of magnetic force as representations of the magnetic power; not merely in the points of quality and direction, but also in quantity. The necessity I was under of a more frequent use of the term in some recent researches, has led me to believe that the time has arrived when the idea conveyed by the phrase should be stated very clearly, and should also be carefully examined, that it may be ascertained how far it may be truly applied in

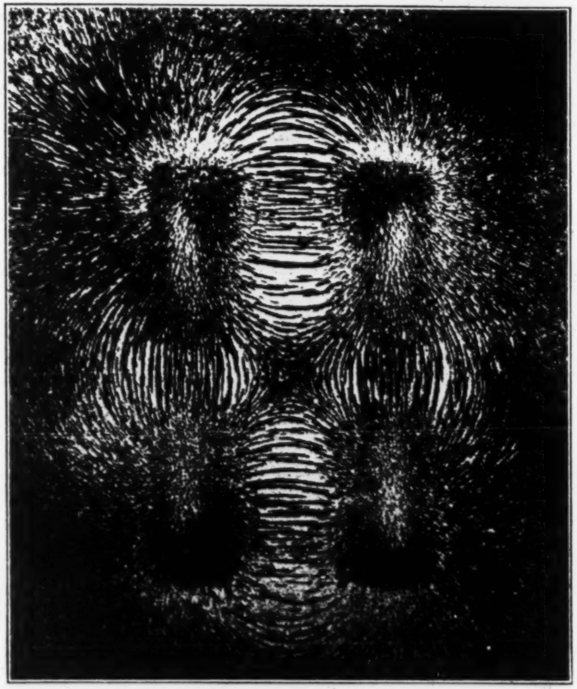


FIG. 4.—Lines of Magnetic Force Produced by Two Straight Parallel Magnets.

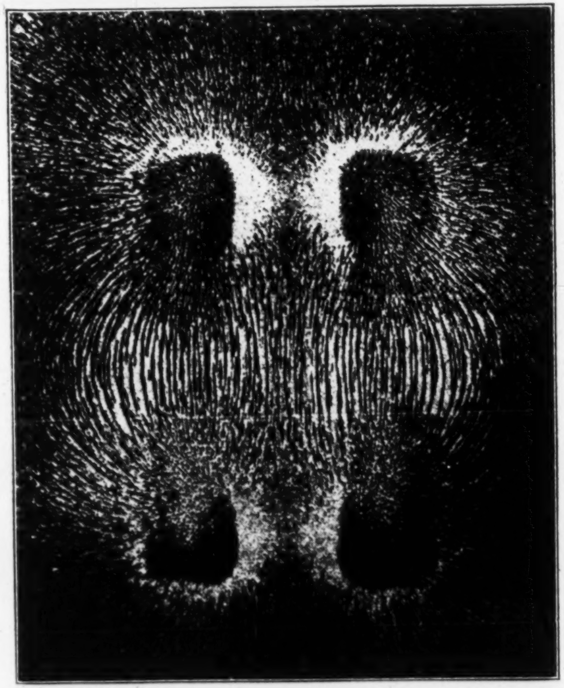


FIG. 5.—Lines of Magnetic Force Produced by Two Straight Parallel Magnets.

Acknowledgment is due to Mr. T. R. Watts of the Research Laboratories of the Westinghouse Electric and Manufacturing Co. for having kindly furnished the photographs for these figures.

representing magnetic conditions and phenomena; how far it may be useful in their elucidation; and, also, how far it may assist in leading the mind correctly on to further conceptions of the physical nature of the force, and the recognition of the possible effects, either new or old, which may be produced by it.

"Now it appears to me that these lines may be employed with great advantage to represent the nature, condition, direction and comparative amount of the magnetic forces; and that in many cases they have, to the physical reasoner at least, a superiority over that method which represents the forces as concentrated in centers of action, such as the poles of magnets or needles; or some other methods, as, for instance, that which considers north or south magnetisms as fluids diffused over the ends or amongst the particles of a bar. No doubt, any of these methods which does not assume too much, will, with a faithful application, give true results; and so they all ought to give the same results as far as they can respectively be applied. But some may, by their very nature, be applicable to a far greater extent, and give far more varied results, than others.

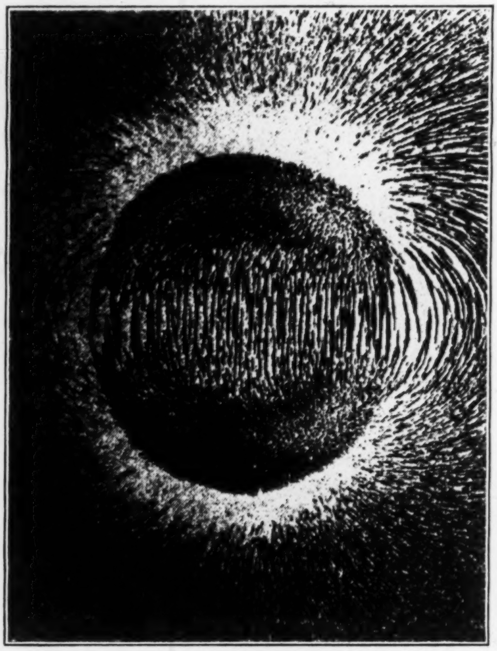


FIG. 6.—Lines of Magnetic Force Produced by a Magnetized Ring.

**Acknowledgment is due to Mr. T. R. Watts of the Research Laboratories of the Westinghouse Electric and Manufacturing Co. for having kindly furnished the photograph for this figure.**

"I have recently been engaged in describing and defining the lines of magnetic force, *i. e.*, those lines which are indicated in a general manner by the disposition of iron filings or small magnetic needles, around or between magnets; and I have shown, I hope satisfactorily, how these lines may be taken as exact representants of the magnetic power, both as to disposition and amount; . . . . The definition then given had no reference to the physical nature of the force at the place of action, and will apply with equal accuracy whatever that may be; and this being very thoroughly understood, I am now about to leave the strict line of reasoning for a time, and enter upon a few speculations respecting the physical character of the lines of force, and the manner in which they may be supposed to be continued through space. We are obliged to enter into such speculations with regard to numerous natural powers, and, indeed, that of gravity is the only instance where they are apparently shut out.

"It is not to be supposed for a moment that speculations of this kind are useless, or necessarily hurtful, in natural philosophy. They should ever be held as doubtful, and liable to error and to change; but they are wonderful aids in the hands of the experimentalist and mathematician. For not only are they useful in rendering the vague idea more clear for the time, giving it something like a definite shape, that it may be submitted to experiment and calculation; but they lead on, by deduction and correction, to the discovery of new phenomena, and so cause an increase and advance of real physical truth, which, unlike the hypothesis that led to it, becomes fundamental knowledge not subject to change. Who is not aware of the remarkable progress in the development of the nature of light and radiation in modern times, and the extent to which that progress has been aided by the hypotheses both of emission and undulation? Such considerations form my excuse for entering now and then upon speculations; but though I value them highly when cautiously advanced, I consider it as an essential character of a sound mind to hold them in doubt; scarcely giving them the character of opinions, but esteeming them merely as probabilities and possibilities, and making a very broad distinction between them and the facts and laws of nature." (Michael Faraday, "Experimental Researches in Electricity," Vol. 3, 1855, g. 34. "On lines of magnetic force," . . . pages 328, 329, 407 and 408.)

It is perhaps not less instructive to quote another great physicist, whose thoughts were influenced by these attempts:

". . . before I began the study of electricity I resolved to read no mathematics on the subject till I had first read through Faraday's 'Experimental Researches in Electricity.' I was aware that there was supposed to be a difference between Faraday's way of conceiving phenomena and that of the mathematicians, so that neither he nor they were satisfied with each other's language. I had also the conviction that this discrepancy did not arise from either party being wrong. I was first convinced of this by Sir William Thompson, to whose advice and assistance, as well as to his published papers, I owe most of what I have learned on the subject.

"As I proceeded with the study of Faraday, I perceived that this method of conceiving the phenomena was also a mathematical one, though not exhibited in the conventional form of mathematical symbols. I also found that these methods were capable of being expressed in the ordinary mathematical forms, and thus compared with those of the professed mathematicians.

"For instance, Faraday, in his mind's eye, saw lines of force traversing all space where the mathematicians saw centers of force attracting at a distance: Faraday saw a medium where they saw nothing but distance: Faraday sought the seat of the phenomena in real actions going on in the medium, they were satisfied that they had found it in a power of action at a distance impressed on the electric fluids.

"When I had translated what I considered to be Faraday's ideas into a mathematical form, I found that in general the results of the two methods coincided, so that the same phenomena were accounted for, and the same laws of action deduced by both methods, but that Faraday's methods resembled those in which we begin with the whole and arrive at the parts by analysis, while the ordinary mathematical methods were founded on the principle of beginning with the parts and building up the whole by synthesis.

"I also found that several of the most fertile methods of research discovered by the mathematicians could be expressed much better in terms of ideas derived from Faraday than in their original form.

"The whole theory, for instance, of the potential, considered as a quantity which satisfies a certain partial differential equation, belongs essentially to the method which I have called that of Faraday. According to the other method, the potential, if it is to be considered at all, must be regarded as the result of a summation of the electrified particles divided each by its distance from a given point. Hence many of the mathematical discoveries of Laplace, Poisson, Green and Gauss find their proper place in this treatise, and their appropriate expressions in terms of conceptions mainly derived from Faraday." (J. C. Maxwell, Preface to his "Treatise," 3rd Edition, Vol. 1, 1892, p. VIII.)

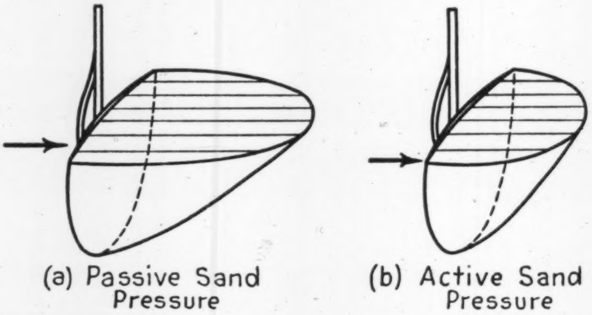
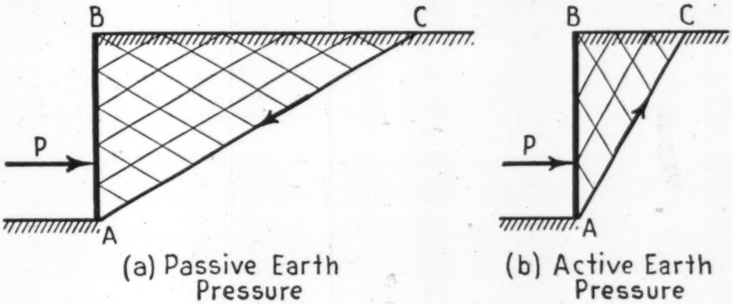

(a) Passive Sand Pressure
(b) Active Sand Pressure

FIG. 7.—Showing the Passive and Active Sand Pressures on a Shovel.

## IDEAL LOOSE MATERIAL

To arrive at a more precise definition of the surfaces of slip, let us consider first the mechanical conditions which must exist in the interior of a loose material, for example, under the surface of a sand dune piled up by the wind on a seashore, or just below the ground of the elevated parts of a wide and gently rising beach, where the summer sun has had ample time to dry out the sand to a considerable depth. Let us take a shovel and experiment a little with this ideal material by sticking it in the sand in a vertical position and trying to move it forward or backward (Fig. 7 (*a*) and (*b*)). Concentrating the attention only on what occurs on the front side of the shovel, by making its back side as free from sand as possible with our hands, we will observe that if the shovel is pushed forward (Fig. 7 (*a*)) in a horizontal direction, it will detach a larger portion of the sand than if it is moved in the opposite direction (Fig. 7 (*b*)), thus gently releasing a portion of the sand on its front side which will then sink under its weight by a small amount. We shall also feel that the force required to move the sand on the front side of the shovel is considerably larger in the first case than in the second (this is indicated by the arrows in Figs. 7 (*a*) and (*b*)). In both cases a certain portion of sand will be detached from the surrounding material along a surface beginning at the edge of the shovel. This surface will be found in the first case to be less steeply inclined to the horizontal plane than in the second. The conditions of the equilibrium in the interior of the detached portion of sand might be described more precisely by considering a plane horizontal ground, extending on one side of a perfectly smooth vertical plate. When this latter is moved in a horizontal direction (Figs. 8 (*a*) and (*b*)) a wedge of the loose material *ABC* bounded by an oblique plane *AC* will be sheared off the ground. The oblique plane *AC* will be inclined less steeply if the wall is displaced towards the mass and more steeply if moved in the opposite direction. If one thinks of the loose material becoming solid for an instant on both sides of the plane *AC*, then in the first case a horizontal force *p* is required to push the detached portion of earth or sand along the oblique section *AC* and to raise it against the action of the frictional forces, while in the second case the frictional forces will support the weight of the wedge *ABC* when it starts to glide down on the inclined plane (Fig. 8 (*b*)). These two ideal cases of equilibrium in loose masses, known in Rankine's theory of earth pressure as the cases of the "*passive*" and of "*active*" earth pressure, may help to clarify conditions which will exist in solid materials when these are brought into the plastic state under a system of stress. The limiting states of equilibrium in loose materials are determined by passive, resisting forces of the type of solid friction,



(a) Passive Earth Pressure

(b) Active Earth Pressure

FIG. 8.—State of Equilibrium According to Rankine. Surfaces of slip in a loose mass held by a perfectly smooth vertical wall.

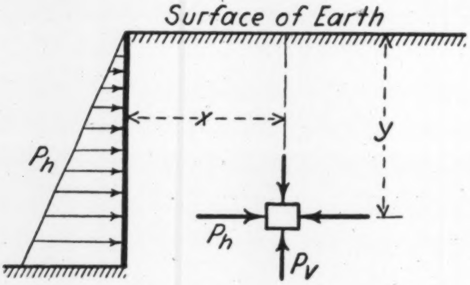

Surface of Earth
x
y
Ph
Ph
Pv

FIG. 9.—Showing the Principal Stresses Under Horizontal Ground.

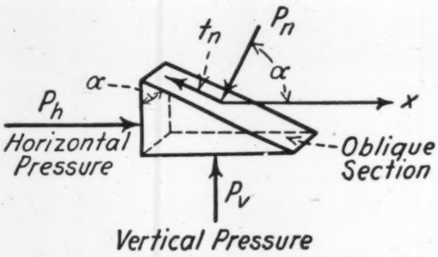

tn
Pn
α
α
x
Ph
Horizontal Pressure
Oblique Section
Pv
Vertical Pressure

FIG. 10.—Showing the Stress Acting on a Small Prism.

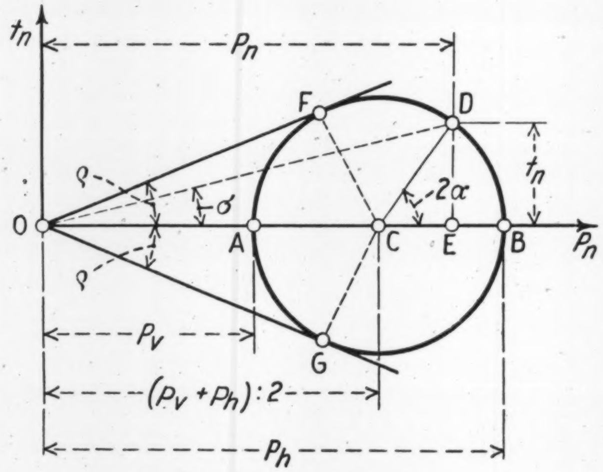

tn
Pn
F
D
ρ
σ
2α
tn
O
A
C
E
B
Pn
ρ
Pv
G
(Pv + Ph):2
Ph

FIG. 11.—Mohr's Circle of Stress.

$p_n$ = Normal Stress.
$t_n$ = Shearing Stress.

always acting in a tangential direction along possible slip planes. For this reason these tangential resisting forces in an ideal loose material can never become greater than a certain fraction of the normal pressure acting along the same sections. As across an arbitrary infinitesimal plane section in the interior of any solid body a certain normal force per unit of area or a normal stress $\sigma_n$ and a certain tangential force per unit of area or a shearing stress $\tau_n$ act, the limiting condition of internal equilibrium in a loose material will be, that whenever movement or slip starts along a definite plane, the shearing stress $\tau_n$ in this plane will just become equal to $\mu\sigma_n$, where $\mu$ is a coefficient of friction. The subscript $n$ designates the direction of the normal $n$ to the plane of the section. In all other plane sections, the shearing stress $\tau_n$ will remain smaller than $\mu\sigma_n$. Instead of $\mu$ the angle of friction $\rho$ might also be introduced, determined by the relation

$$\tan \rho = \mu.$$

In an ideal homogeneous heavy mass of a loose material bounded by a horizontal plane and supported on one side by a perfectly smooth vertical wall, the distribution of the pressures along any vertical plane parallel to the wall must be the same. Along such planes as well as along horizontal planes in the interior of the mass no reason can be found for the existence of shearing stresses. Hence these planes will be principal planes of stress; that is, such planes along which only normal pressures will act. As we are here only investigating the effects of a small horizontal displacement of the vertical wall in a direction perpendicular to its plane, only such planes of slip will be produced which are perpendicular to the plane of the figure (Figs. 8 (*a*) and (*b*)). The state of stress existing in an element of the mass at a certain depth $y$ (Fig. 9) below the horizontal bounding plane will be given by (1) a certain pressure $p_h$ acting in a horizontal direction and perpendicularly to the plane of the wall; (2) a certain pressure acting in the vertical direction $p_v$; (3) a pressure $p_3$ acting in the lateral horizontal direction. The absolute value of $p_3$ is always found lying between the values of $p_h$ and $p_v$. This intermediate principal pressure $p_3$ will therefore have no appreciable influence on the conditions under which slip will occur.

In an oblique plane whose normal $n$ makes an angle $\alpha$ with the $x$ axis (Fig. 10) (this latter assumed in a direction perpendicular to the wall) a normal pressure $p_n$ and a shearing stress $t_n$ will act:

$$\left.\begin{aligned} p_n &= \frac{p_h + p_v}{2} + \frac{p_h - p_v}{2} \cos\ 2\alpha \\ t_n &= \frac{p_h - p_v}{2} \sin\ 2\alpha \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

or after eliminating $\alpha$ from these equations, according to O. Mohr:

$$\left(p_n - \frac{p_h + p_v}{2}\right)^2 + t_n^2 = \frac{(p_h - p_v)^2}{4} = \text{const} \ldots\ldots\ldots\ldots\ldots (2)$$

This last relation, in which the quantities $p_n$ and $t_n$ appear as the rectangular coordinates of a point $D$ situated on a circle having $\frac{p_h - p_v}{2}$ as its radius (Mohr's circle) (Fig. 11) serves to determine the ratio of the shearing to the normal stress, $t_n:p_n$, for any value of the angle $\alpha$. From Eq. 1 it follows that the radius $CD$ must be inclined at an angle $2\alpha$ to the line $OC$. The ratio $t_n:p_n$ corresponding to an angle $\alpha$ is equal to $ED:OE$ or also equal to the tangent of the angle $\sigma$ shown in Fig. 11. According to the preceding this ratio $t_n:p_n$ attains its largest value $= \mu = \tan \rho$ along the slip planes. From Mohr's circle it follows, therefore, that the greatest value of the ratio

$$t_n:p_n = \tan \rho \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

will be obtained by drawing from the origin $O$ two tangent lines to the circle. Hence these two lines determine the positions of the slip planes. From the fact that two such tangent lines can be drawn, it follows, furthermore, that at an arbitrary point in a loose material *two slip planes must always exist.* Their positions are determined by the angles of the radii $CF$ and $CG$. The angles under which the planes of slip must be inclined with respect to the principal pressures $p_v$ or $p_h$, respectively, will be equal to the halves of the angles $ACF$ and $FCB$ respectively, or equal to:

$$\left.\begin{aligned} \frac{1}{2}\left(\frac{\pi}{2} - \rho\right) &= \frac{\pi}{4} - \frac{\rho}{2} \\ \frac{\pi}{2} - \frac{1}{2}\left(\frac{\pi}{2} - \rho\right) &= \frac{\pi}{4} + \frac{\rho}{2} \end{aligned}\right\} \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

The critical values of the normal and shearing stress $p_n$ and $t_n$ which will act across these two slip planes are given by the abscissæ and ordinates of the points of tangency $F$ and $G$ (Fig. 11). Hence we arrive at the result that at any point in the interior of a heavy, granular mass in the limiting state of equilibrium at which slip or movement just starts, *two planes of slip form*, symmetrically inclined at equal angles to the directions of the major or the minor pressure, whose planes intersect along the direction of the intermediate principal pressure $p_3$. If $p_1$ designates the major and $p_2$ the minor principal pressure at a point, it furthermore follows that in the state of stress that has been described as the passive state of pressure behind a perfectly smooth retaining wall, the major principal pressure $p_1$ will be oriented in the horizontal, while in the case of the active pressure, in the vertical position (see Fig. 12).

The two planes of slip always form an angle with the major principal pressure $p_1$ smaller than 45 deg. The stress circles (such as the one shown in Fig. 11) representing all the possible limiting states of stress in different elements of the loose material obviously must be a family of circles (Fig. 13) having their centers $C$ on the $p_n$ axis and the two straight lines $OF$ and $OG$ as their envelopes.

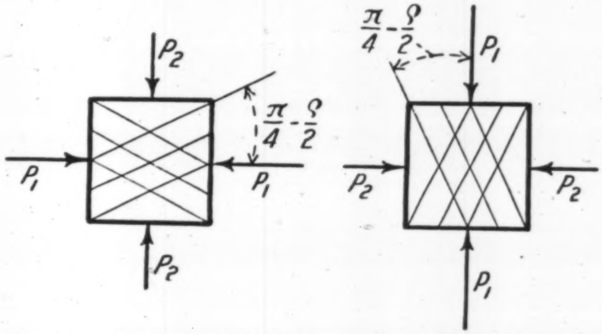

$P_2$
$\frac{\pi}{4}-\frac{\rho}{2}$
$P_1$
$P_1$
$P_2$
$\frac{\pi}{4}-\frac{\rho}{2}$
$P_1$
$P_2$
$P_2$
$P_1$

FIG. 12.—Position of Slip Planes When Major Principal Pressure $p_1$ Is Horizontal and When It Is Vertical.

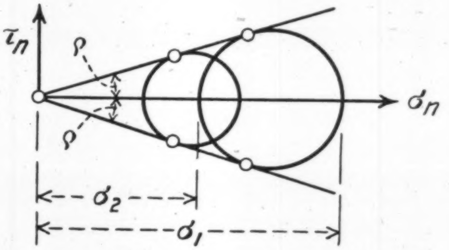


FIG. 13.—Showing Stress Circles for Limiting States of Equilibrium in Loose Material.

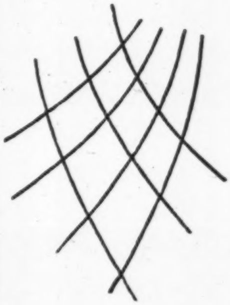


FIG. 14.—Showing Surfaces of Slip.

## PLANE STRESS IN A LOOSE MATERIAL

A more general case of great practical interest is the state of stress in a loose material which depends on two coordinates; for example, the rectangular coordinates $x$ and $y$, both measured in a vertical plane, the first in the horizontal, the second in the vertical direction downward

(Fig. 9). An example of such a state of plane stress is the equilibrium of a loose material supported laterally by a *rough* retaining wall. More exact observations of slides of earth indicate that sliding frequently occurs along curved surfaces (Fig. 14). The great landslides which have been observed in connection with excavations for canals were described as occuring along such curved surfaces. Theory and observations both indicated that in an ideal loose material, in which a plane state of stress occurs, *two families of curved surfaces of slip must exist* (Fig. 14), intersecting each other everywhere at the same angle, $\frac{\pi}{2} - \rho$. F. Kötter[1] and H. Reissner[2] have expressed the sufficient conditions which must hold in these cases of equilibrium. A plane state of stress is determined by three components of stress; that is, if rectangular coordinates $x$ and $y$ are used, by a normal stress $\sigma_x$ acting parallel to the $x$ axis, by a normal stress $\sigma_y$ acting parallel to the $y$ axis and by a shearing stress $\tau$. If $\sigma_1$ and $\sigma_2$ are the principal stresses and $\alpha$ designates the angle of $\sigma_1$ with the $x$ axis, then according to Eq. 1:

$$\left.\begin{aligned} \sigma_x &= \frac{\sigma_1 + \sigma_2}{2} + \frac{\sigma_1 - \sigma_2}{2} \cos 2\,\alpha \\ \sigma_y &= \frac{\sigma_1 + \sigma_2}{2} - \frac{\sigma_1 - \sigma_2}{2} \cos 2\,\alpha \\ \tau &= \frac{\sigma_1 - \sigma_2}{2} \sin 2\,\alpha \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

According to what has been said before, the condition that in all small elements of the loose material limiting states of equilibrium must exist, is expressed by a relation between $\sigma_1$ and $\sigma_2$:

$$\sin \rho = \frac{\sigma_1 - \sigma_2}{\sigma_1 + \sigma_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

This relation expresses the fact that the principal stress circles must have for their common envelopes two straight lines inclined at the angle of friction $\rho$ to the $x$ axis (Fig. 13). If

$$\sigma = \frac{\sigma_1 + \sigma_2}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

using this last relation it follows that:

$$\sigma_1 - \sigma_2 = (\sigma_1 + \sigma_2) \sin \rho = 2\,\sigma \sin \rho \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

---

[1] *Berliner Berichte*, 1903.

[2] *Enzyklopaedie der Mathematischen Wissenschaften*, Vol. 4, p. 4; also later publications.

The three components of stress $\sigma_x$, $\sigma_y$, and $\tau$ can be expressed also by:

$$\left.\begin{aligned} \sigma &= \sigma\,(1 + \sin\rho\cos 2\,\alpha) \\ \sigma_y &= \sigma\,(1 - \sin\rho\cos 2\,\alpha \\ \tau &= \sigma\sin\rho\sin 2\,\alpha \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

We have seen that in each point $x$, $y$ two planes of slip form inclined at an angle $\frac{\pi}{4} - \frac{\rho}{2}$ to the major principal pressure $\sigma_1$. The angle $\alpha$ in Eq. 9 may therefore be replaced by the angle $\beta$ under which one of *the slip planes* (the "first slip plane") is inclined to the $x$ axis, as

$$\beta = \alpha + \frac{\pi}{4} - \frac{\rho}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

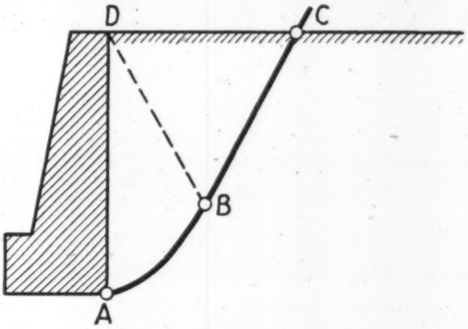


FIG. 15.—Showing Surface of Slip *ABC*, Limiting Wedge of Earth, *ABCD*, Supported by a Rough Retaining Wall. (According to T. von Karman.)

Hence

$$\left.\begin{aligned} \sigma_x &= \sigma\,[1 + \sin\rho\sin(2\,\beta + \rho)] \\ \sigma_y &= \sigma\,[1 - \sin\rho\sin(2\,\beta + \rho)] \\ \tau &= -\,\sigma\sin\rho\cos(2\,\beta + \rho) \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

By combining these last three equations with the two conditions of equilibrium:

$$\frac{\partial\sigma_x}{\partial x} + \frac{\partial\tau}{\partial y} = 0,\ \frac{\partial\sigma_y}{\partial y} + \frac{\partial\tau}{\partial x} = \gamma \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

valid in a state of plane stress (where $\gamma$ is the weight per unit of volume of the loose material), the five necessary equations have been found which will determine the five unknown functions $\sigma_x$, $\sigma_y$, $\tau$, $\sigma$, $\beta$. This theory leads to two families of curved surfaces of slip. Each of these two families of surfaces intersects the surfaces of the other family at a constant angle $\frac{\pi}{4} - \rho$ so that they both form, in their intersection curves with the $x$, $y$ plane, a system of isogonal trajectories. In some few special cases, it was possible to construct these curves or the exact shape of the two families of surfaces of slip, and comparison with actual observations of the earth movements seem to indicate that by these means conditions as they may prevail in loose earth supported by a rough retaining wall have adequately been expressed. According to T. von Karman, in the case of a rough vertical retaining wall the main surface of slip *ABC* along which the wedge of earth would tear off is found to consist of a partly curved portion *AB* and a straight portion *BC* (see Fig. 15).

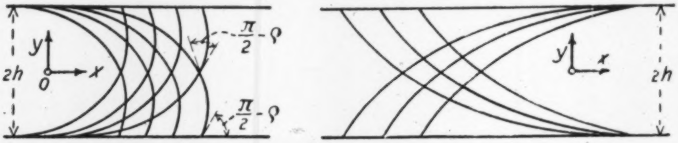


FIG. 16.—Showing Families of Isogonal Cycloids as Slip Lines. (According to Hartmann.)

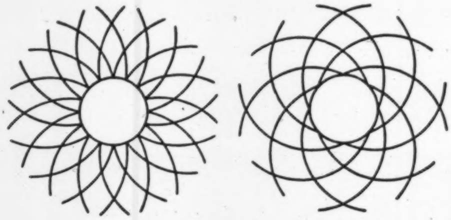


FIG. 17.—Showing Families of Isogonal Logarithmic Spirals as Slip Lines. (According to Hartmann.)

## ISOGONAL SURFACES OF SLIP IN A LOOSE MATERIAL

If the term $\gamma$ in Eq. 12 expressing the gravity forces is neglected, the five equations of Eqs. 11 and 12 might be integrated in a general way. Thus, in several special cases of plane equilibrium the exact equations of the surfaces of slip could be derived. A number of interesting examples constructed by W. Hartmann[1] are shown in Figs. 16 and 17 by means of their slip lines.

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[1] With regard to this theory see "Handbuch der Physik," by Geiger and Scheel, Vol. VI, article on Plasticity, p. 495, J. Springer, Berlin, 1928.

## SLIP IN PLASTIC MATERIALS

As has already been indicated, very analogous phenomena to the formation of the surfaces of slip in loose materials have been observed in solid metals and in certain rocks. If a test specimen of mild steel is polished on the surface until it has a mirror-like finish, and subjected to a tensile load or to compression, very regular parallel markings might be seen to appear quite suddenly on the surfaces when the stresses just reach the limit of plasticity or the yield point (Fig. 18).

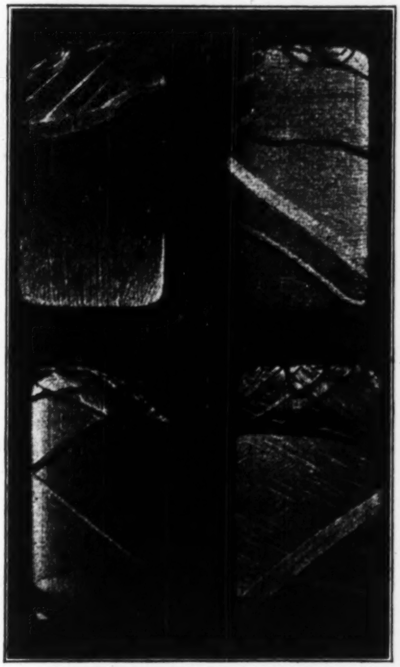


FIG. 18.—Strain or Flow Figures (Lüders' Lines) on Mild Steel.

A closer observation shows that these markings are the intersections of certain thin layers with the surface of the test specimen in which the material has been plastically deformed locally. The most remarkable property of these figures is their close relation to the states of stress under which they were produced. In a prismatic mild steel specimen subjected to axial tension or compression, the plastic layers are inclined at an angle of approximately 45 deg. They can frequently be seen to form two surfaces which intersect each other at approximately right angles. These markings may be produced under various stress conditions and if the states of stress are symmetrical these flow layers may form patterns of astonishingly regular design on the surfaces of the test specimens.[1]

The structural changes due to the plastic deformations of the crystal grains or in single crystals of a metal have been carefully described in recent years. The mechanism of the plastic deformations of crystals is also known to consist in gliding movements of one part of a crystal relative to the other. While the plastic deformations in the crystal grains or in single crystals appear to consist of small plane displacements, detectable by means of a microscope and closely related to the crystalline symmetry, the surfaces of slip in the phenomenon with which we are concerned here extend over a large number of crystal grains. It is perhaps worthy of mention that such a remarkable analogy seems to exist between two mechanisms and that to the infinitesimal gliding movements in the crystal grains in some respect quite analogous phenomena seem to correspond in the formation of the surfaces of slip and of the plastic layers observable on a large scale in polycrystalline materials.

## SCHLIEREN METHOD FOR OBSERVING THE FLOW-LAYERS IN PLASTIC MATERIALS

Through the courtesy of the Research Laboratories of the Westinghouse Electric and Manufacturing Co. a new optical bench has been constructed recently which may serve to demonstrate and to reproduce these regular markings on the polished surfaces of plastic materials with a high degree of exactness. The first observations by means of the method described here were made several years ago, when the speaker on a suggestion of L. Prandtl tried to apply the Schlieren method for this purpose.[2]

---

[1] These lines known to engineers as *flow or strain figures* or as "*Lüders' lines*" quickly spread over the surface of mild steel specimen stressed to the yield point. Attention to their regular appearance seems to have been called first by Lüders in Dingler's *Polytechn. Journal*, 1854. In France they are also known as "*lignes de Piobert.*" The French artillery officer L. Hartmann appears to have been the first who studied their laws of formation thoroughly in 1896. Their importance for the mechanics of the plastic state of metals was recognized earlier however (in 1882) by Otto Mohr (*Zeitschrift des Vereines Deutscher Ingenieure* 1900). Recently they were the subject of several investigations (by T. H. Turner, J. D. Jevons, E. W. Fell, A. Fry and others). These figures have occasionally also been called "force lines" ("Kraftwirkungslinien") but it should be borne in mind that they neither indicate the direction of forces in a field nor can they be utilized for a quantitative measurement of forces, because a state of *stress* in general cannot be expressed by the field of a vector, but by that of a vectorial quantity of second order, namely, a tensor (or a linear vector function) depending on six components, or on *two* vectors.

[2] The "Schlieren" method has heretofore been used only to observe the changes in density or refractive power in transparent bodies; for example, to make visible the "Schlieren," that is, the invisible inclusions in glass with slightly different optical properties (a slightly different index of refraction) which may disturb the optical qualities of lenses. By means of this method E. Mach has made visible sound vibrations in air, and the compression waves produced by a projectile of a gun, moving with a velocity higher than that of sound in air. Recently the method has been applied to study the flow of gases moving with a velocity higher than that of sound, utilizing the changes in density and thus in refractive power of gases.

To observe something on the highly polished surface of an opaque piece of material, which itself reflects all the light, represents a contradiction in itself. Slight disturbances on a highly polished metal can only be observed by reflecting in it other objects in space; for example, the wooden frame or cross of a window, when holding the piece against the light entering through the window of a room. The Schlieren method consists of an arrangement of lenses as indicated in the schematic diagram, Fig. 19. A small circular opening on the side of a prism, *P*, turned towards the test specimen, *T*, is assumed as the source of light which serves to illuminate the test specimen *T*. (This latter is of course turned with its polished surface towards the prism.) By means of the lens *L* and the reflecting mirror surface of the test specimen *T*, an image of the bright circular opening of *P* is thrown at a certain distance. Where this image appears, an opaque screen or diaphragm, *D*, might be inserted in the passage of the light beam. If the reflecting surface of the test specimen is a perfect mirror, it will

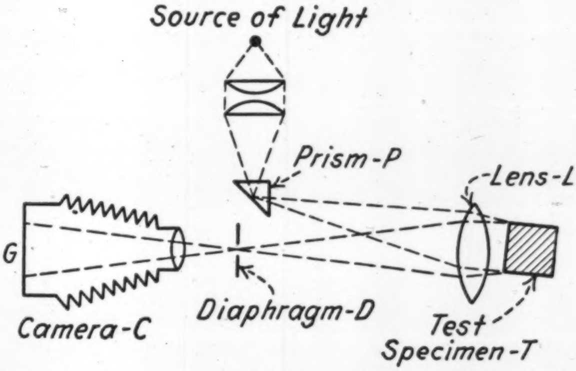


FIG. 19.—Schlieren Method for Observing Plastic Deformations.

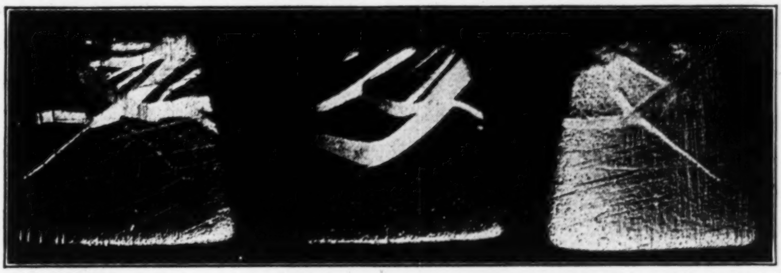


FIG. 20.—Lüders' Lines on Mild Steel Observed by Schlieren Method. (Compression Tests.)

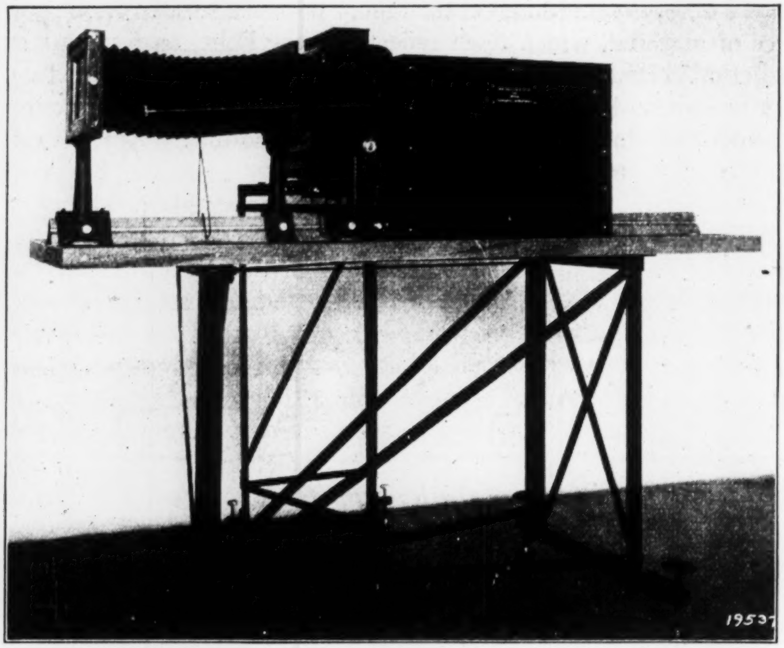


FIG. 21.—New Optical Bench for Observing Strain or Flow Figures.

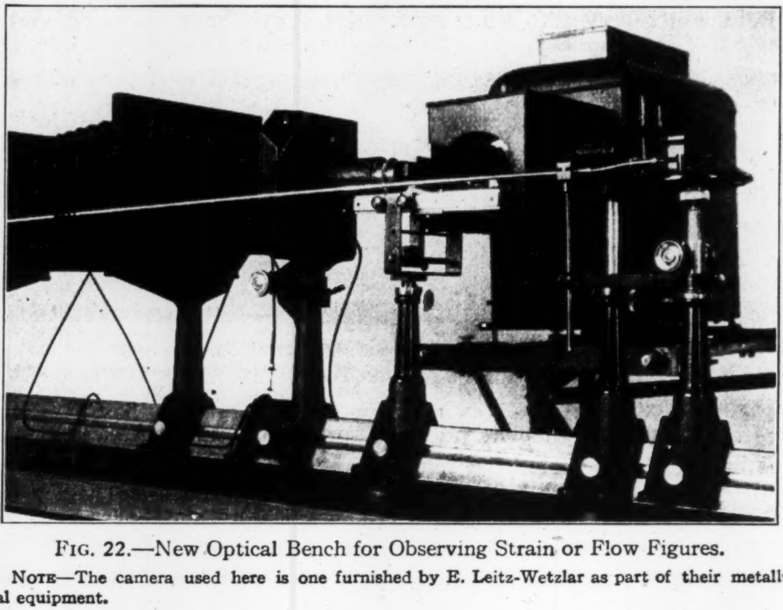


FIG. 22.—New Optical Bench for Observing Strain or Flow Figures.

**NOTE**—The camera used here is one furnished by E. Leitz-Wetzlar as part of their metallurgical equipment.

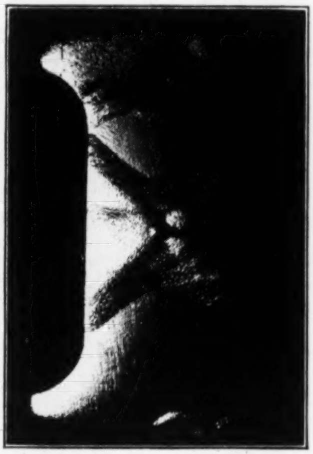


FIG. 23.—Showing Flow Layers on Steel.

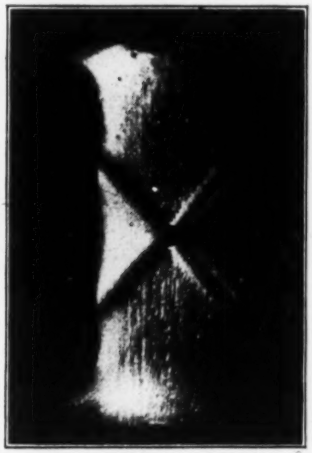


FIG. 24.—Showing Flow Layers on Hard Copper.

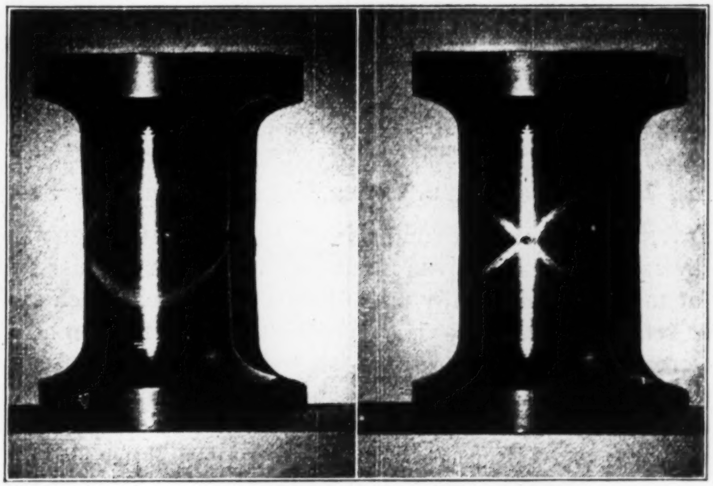


FIG. 25.—Paraffin Compression Cylinder.

produce a perfect image of the opening of $P$ at $D$ and observing the whole from the side of the ground glass of a photographic camera the face of the test specimen $T$ will appear uniformly illuminated. If an opaque screen is inserted at the converging place of the light beams at $D$, the light coming from the test specimen will suddenly be cut off and this will appear dark. However, if slight disturbances are present on the highly polished front surface of the test specimen, such as those produced by minute scratches of the polishing machine or also by the slight plastic deformations of flow layers intersecting the surface of the mirror, the light coming from these places will not converge perfectly at $D$. Some will pass around $D$, others will be shut off. The result will be that these irregularities on the polished surface will now become visible on the ground glass of the camera. Inserting the opaque screen at $D$ will therefore bring out at once all the details on the polished surface, which would all have remained invisible had the lens $L$ and the diaphragm $D$ not been applied.

While the method described is particularly fitted to make visible the slightest disturbances in a plane surface, changes in structure occurring in the plastic layers may also be studied in the interior of test specimens of steel by etching. The flow layers in mild steel, for example, might be made visible in a cross-section of a test specimen by first tempering this latter after it has been deformed at about 200° C. and then etching it with the solutions of A. Fry. The flow layers appear as black lines or strips in the cross-sections and might be brought out on these after deformation in a manner not unlike that in which the latent picture is developed on a photographic plate after an exposure.

## FLOW LAYERS PRODUCED UNDER VARIOUS STATES OF STRESS

Provided that certain conditions are satisfied, the pair of latent flow layers existing at a given point in a material that just starts to yield or to deform plastically might be made visible. Such examples can be seen in Figs. 23 to 31. Figure 23 shows two broad plastic layers crossing in the middle of the front side of a mild steel specimen containing a hole, which was tested in axial compression.

Figure 24 indicates the same phenomenon on a compression specimen of hard, cold-worked copper.

Figure 25 shows the front and the side view of a dark paraffin cylinder in which two crossing flow layers (appearing white in the photographs) were produced by axial compression. The artificial formation of a pair of flow layers at a given point of a test specimen is facilitated if this contains a small hole and the material to be tested has a sharply

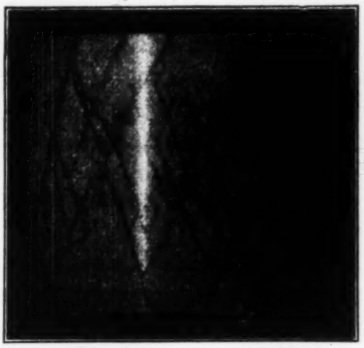


FIG. 26.—Showing Slip Lines on a Marble Cylinder. (Compression test by Prandtl-Rinne.)

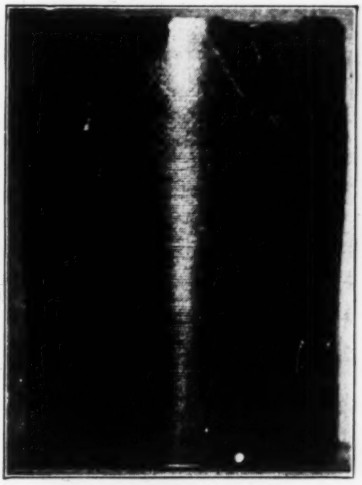


FIG. 27.—Paraffin Compression Test.

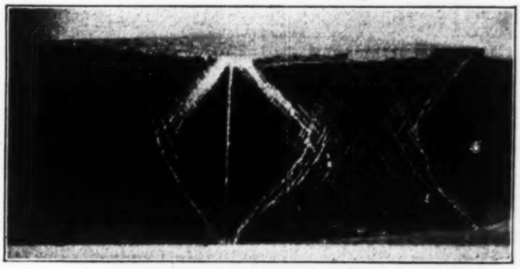


FIG. 28.—Showing Slip Lines Produced on Paraffin Prisms Tested in Compression.

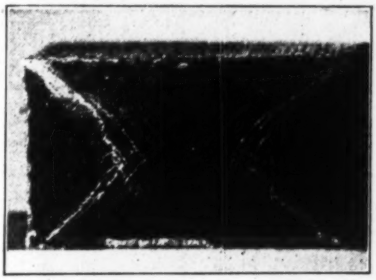


FIG. 29.—Showing Slip Lines Produced on Paraffin Prisms Tested in Compression.

defined yield point or plastic limit. As already mentioned, the flow layers in mild steel cross at an angle which does not differ much from a right angle. Hard copper or soft paraffin behave in a similar manner. However, in other materials, especially in certain brittle rocks which do not immediately fracture in a compression test, flow layers may be made visible which differ considerably in their position to the axis of principal or major compression. Such an example can be seen in Fig. 26 which is a view taken after a compression test of a marble cylinder with a highly polished surface. The dark markings on this surface are the traces of the surfaces of slip on the former. They form a pattern of two systems of helixes crossing at a constant angle considerably less than 90 deg. The angle of pitch of these helixes with the generatrices of the cylinder (that is with the direction of major principal compression) is considerably smaller than 45 deg.

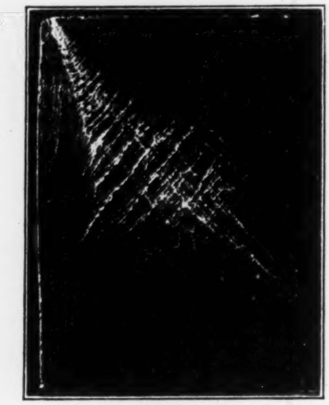


FIG. 30.—Showing Slip Lines Produced on Paraffin Prisms Tested in Compression.

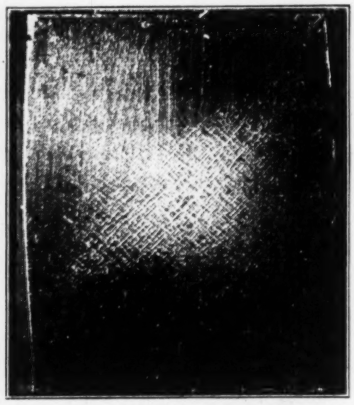


FIG. 31.—Showing Slip Lines Produced on Paraffin Prisms Tested in Compression.

Figure 27 shows a paraffin cylinder tested under similar conditions, in which the helixes again are visible, crossing now however at nearly right angles.

In Figs. 28 to 31 can be seen the slip lines on prisms of paraffin with a square cross-section tested in compression in an axial direction between rigid steel plates. The white patterns appearing on the sides of the compression prisms disclose very regular systems of slip surfaces, the slopes of which are apparently affected by the friction acting in the compression planes. This latter tends to prevent the lateral expansion of the material.

In Fig. 31 a fine network of the slip lines can be seen, which appeared when the sides of the paraffin prism started to bulge out lat-

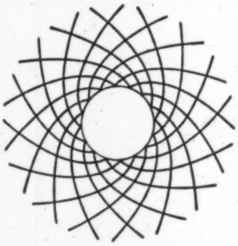
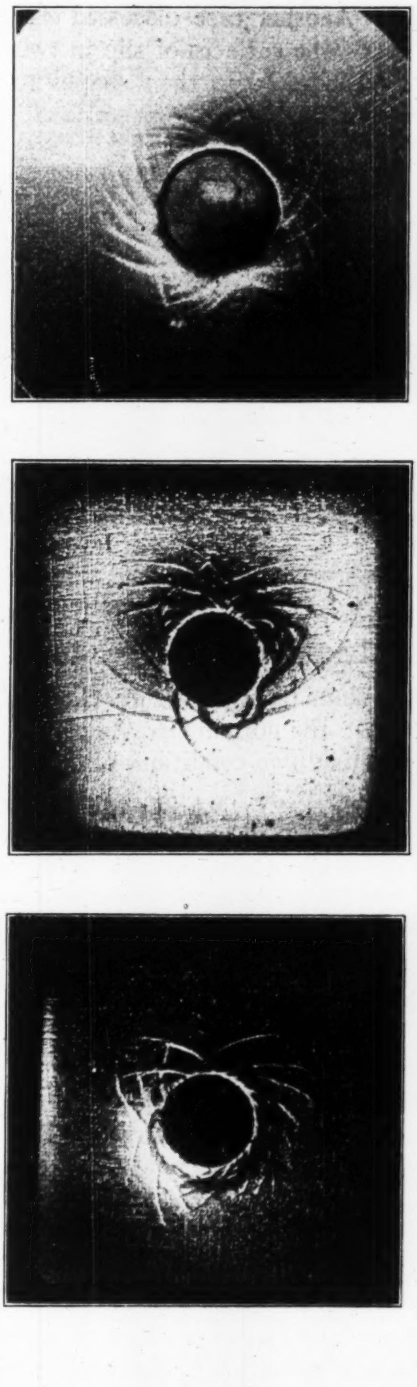


FIG. 32.—Showing Logarithmical Spirals as Slip Lines.



FIGS. 33 to 35.—Showing Logarithmical Spirals as Slip Lines Produced on Mild Steel.

erally. Another case discussed elsewhere in detail is that of plastic torsion. The surfaces of slip in twisted steel bars were found to be closely connected in their shapes to the surfaces of constant slope (corresponding to the cross-sectional figures of the twisted bars).

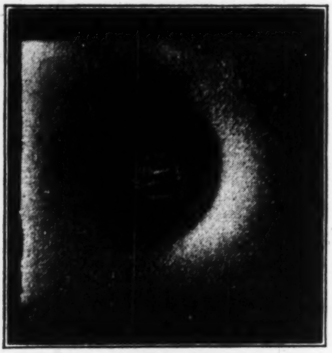


FIG. 36.—Showing an Indentation on Soft Copper. (Photographed by Schlieren Method.)

## SLIP LINES IN PLANE FIELDS OF STRESS IN PLASTIC MATERIALS

A plane plastic state of stress is determined by three components of stress; the normal stresses $\sigma_x$, $\sigma_y$ and the shearing stress $\tau$, which must satisfy two conditions of equilibrium:

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau}{\partial y} = 0, \frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau}{\partial x} = 0 \qquad (13)$$

In case of a slow flow of a perfectly plastic material a third equation is

$$(\sigma_x - \sigma_y)^2 + 4\,\tau^2 = 4\,k^2 = \text{const.} \qquad (14)$$

In this condition of plasticity, $k$ may be taken equal to

$$k = \frac{\sigma_0}{\sqrt{3}} \qquad (15)$$

where $\sigma_0$ designates the yield stress in pure tension or compression. According to R. von Mises[1] and H. Hencky[2], who are given credit for having developed the analysis of these cases of plane plastic flow, the three simultaneous equations, Eqs. 13 and 14, can be integrated in a general way if the surfaces of slip are again introduced as the independent variables.[3] A number of interesting analytical solutions are known. Some examples are shown in Figs. 32, 37 and 38 by their slip lines.

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[1] *Zeitschift für angewandte Mathematik und Mechanik,* Vol. 5, p. 147 (1925).

[2] *Zeitschift für angewandte Mathematik und Mechanik,* Vol. 3, p. 241 (1923).

[3] See "Handbuch der Physik," by Geiger and Scheel, Vol. VI, article on Plasticity, p. 495, J. Springer, Berlin, 1928.

It might be worth while to mention that these families of curves have a number of interesting properties. If they have common envelopes they limit certain regions of space by their enveloping curves and no plastic equilibrium can exist in such cases outside of these regions.[1] In the case of the example shown in Fig. 37, the slip lines consist of two families of orthogonal cycloids.[2] These two examples correspond somewhat to the cases of a soft plastic material squeezed between two rigid, rough, parallel plates and allowed to flow only in one direction. The states of stress in these two cases show a remarkable analogy to the active and to the passive states of earth pressure as described before for a loose material.[1]

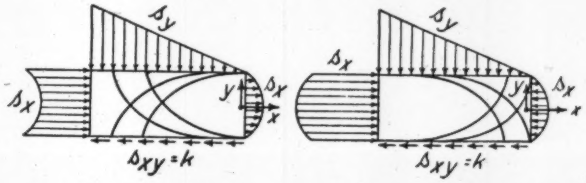

FIG. 37.—Showing Distribution of Stress in Plastic Mass Squeezed Between Rough Parallel Planes. (According to L. Prandtl.)

**The two corresponding systems of slip lines are orthogonal cycloids; left, passive, right, active state of stress.**

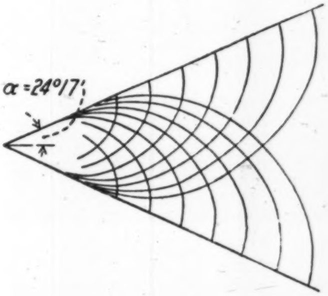

FIG. 38.—Two Orthogonal Families of Slip Lines Having Two Intersecting Straight Lines as Their Envelopes.

## SURFACES OF SLIP IN GEOLOGY

In minerals and in rocks, which were once buried deep under the weight of the overlying strata of rock, most remarkable evidences of severe plastic deformations have been noticed. The mountain building forces together with denudation and erosion have brought such deeply buried strata near or to the surface of the earth, where they are exposed on mountain sides or can be studied in the interior of mines, wells, or in railroad tunnels. These observations have disclosed an abundance of highly interesting and important facts. There were not only found in the structural changes of certain sedimentary or igneous rocks and minerals, traces of various movements which showed a complete analogy to the changes in structure observed in metals after severe plastic deformations, but it was possible in this way for example to describe and analyze more precisely the preceding movements in the strata during past times. In this way W. Schmidt[1] could show from a study of the orientation of the axes of the small quartz crystals contained in certain rocks in the high Tyrolian Alps that in regions many square kilometers in extent the axes of the small quartz crystals pointed in a common direction of the compass. Parallel arrangements of crystal axes in definite directions in metal test specimens, as a consequence of severe plastic deformations such as rolling, are well known to the metallurgist. By similar observations it was possible to determine the position of the principal strain axes in a mountain or the directions of the maximum shearing displacements in the rocks. In igneous rocks or rocks metamorphosed by pressure and heat (presence of volcanic masses) many analogous traces of movements are known to geologists, such as, for example, the regular arrangements of the "Sanidine" plate inclusions in the volcanic trachyte in the "Drachenfels" near Bonn, Germany, described by H. Cloos and W. Riedel, or of the "Schlieren" (the streaks) in large granite stocks (H. Cloos[2]).

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[1] *Zeitschrift für Physik*, Vol. 30, p. 106 (1924).

[2] The components of stress for this case were given by L. Prandtl, *Zeitschrift für angewandte Mathematik und Mechanik*, Vol. 3, p. 401 (1923).

Cases where a layer of a plastic material supports more rigid strata are known to the geologist because to such occurrences might be traced various important effects in geology. A soft layer of a highly plastic material enclosed between strata of a more rigid or more brittle rock (such as sandstone or limestone) may be the cause of very interesting changes in the position of the strata overlying the plastic layer, if the action of gravity and of vertical and lateral forces is considered.

The higher degree of plasticity in a layer can be due to various factors; for example, to the content of water in a layer of clay or to oil in sands, or to the higher temperature of a slowly cooling volcanic rock enclosed between sedimentary layers partly lifted up by the under-flow of a volcanic mass. A further example is a rock-salt layer enclosed between brittle sandstones. The formation of the rock-salt stocks or domes in the north German plains and in the regions of the Gulf of Mexico is attributed according to E. Seidl,[1] E. R. Lilley[2] and others to the high plasticity of salt. The stocks from which salt is carried or in the neighborhood of which in the uplifted adjoining strata the oil accumulates are considered as thickened parts of an originally thin rock-salt layer extending horizontally over large areas. Rock salt has probably a smaller specific gravity than the surrounding rocks and rises from the depth, where through lateral movements in the overlying tables of brittle rocks zones of weakness were formed. Here the salt accumulates to form the salt domes or stocks. We may say that rock salt behaves as a lubricant quite similarly as a layer of soap or paraffin would act, when enclosed between heavy, rigid metal or wood plates. Owing to the mountain building forces, large blocks of the rigid overlying strata may have been somewhat displaced with respect to each other and to the underlying rigid beds. During these movements, the rock salt was squeezed out from below the depth and raised along the zones of weakness nearer to the surface.

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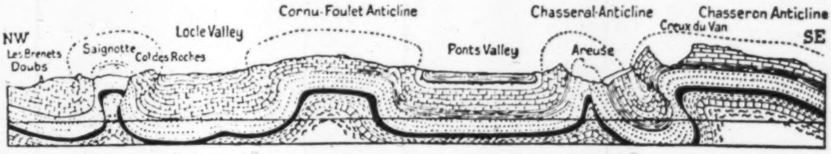
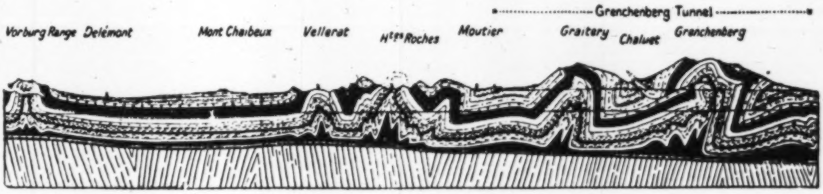
[1] "Gesteinsumformung," F. Deuticke, Leipzig and Vienna, 1925. With reference to these structural changes the reader must be referred to a number of excellent treatments in treatises on geology and mineralogy. A recent one is that by B. Sander, "Gefügekunde der Gesteine," J. Springer, Berlin, 1930, containing most valuable observations.

[2] "Tektonische Behandlung magmatischer Erscheinungen (Granittektonik)," Gebrüder Borntraeger, Berlin, 1925, and a number of other publications.



NW
Les Brenets
Doubs
Saignotte
Col des Roches
Locle Valley
Cornu-Foulet Anticline
Ponts Valley
Chasseral-Anticline
Areuse
Creux du Van
Chasseron Anticline
SE

FIG. 39.—Section Through the Swiss Jura. (According to L. W. Collet, "The Structure of the Alps"; profile by Albert Heim, "Geologie der Schweiz.")



Grenchenberg Tunnel
Vorburg Range
Delémont
Mont Chaibeux
Vellerat
Hte Roches
Moutier
Graitery
Chaluet
Grenchenberg

FIG. 40.—Section Through a More Heavily Folded Portion of the Jura Mountains in Switzerland. (According to Collet; profile by Buxtorf.)

The folding of mountain chains such as those of the Swiss Jura is attributed to a perfectly similar cause. The school of Swiss geologists, with A. Heim, L. Collet, Buxtorf and others, have conclusively demonstrated by means of their carefully prepared profiles of these mountain chains, based on the knowledge attained by the building of long railroad tunnels and by the direct observation that the act of the folding of the Jura chains in a certain sense is a surface phenomenon (Fig. 39). In the cases of the anticlines, where the rocks were raised and bent the most from their formerly horizontal position, the older underlying strata do not appear. Due to the lateral push exerted by the adjoining masses of the high Alps in which innumerable evidences of quite unexpectedly large horizontal overthrusts have been discovered, the upper strata of rocks, of which the Jura chains were originally formed, were detached from the underlying beds. It is even possible that a rock-salt layer or else a layer of a plastic material on which the brittle upper layers were resting helped to detach the folded mountain masses from their unfolded substratum. The profiles taken from the book of L. W. Collet, "The Structure of the Alps" (Edward Arnold and Co., London), indicate clearly this state of affairs. Besides folding, in the Jura Mountains rupture along oblique planes with discontinuities is also observable. One portion of the broken strata is horizontally pushed over the other portion along inclined planes.

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[1] "Die Permische Salzlagerstätte," Berlin, 1914, also various other publications.

[2] "The Geology of Petroleum and Natural Gas," New York, 1928.

The facts leading to the theory of the formation of the "Swiss nappes" ("Deckentheorie") (Theory of Overthrusts in America) are perhaps the most magnificent revelations of these effects of slip on a large scale occurring in nature. In the interior of the highly tangled masses of contorted strata in the main chains of the Alps (and other mountain chains) the geologist has discovered a number of most remarkable surfaces or planes of thrusts. Inexplicable facts such as the occurrence of an isolated fragment of a geologically older stratum on the top of a high mountain, which itself consists of much younger strata or of the overriding of a set of strata on the same set and succession of layers or the occurrence of a portion of an old crystalline rock completely (also from below) enclosed in and surrounded by younger sedimentary layers became comprehensible when the modern School of Geologists had turned to explain these and other similar observations by assuming large horizontal displacements of a magnitude of many miles or so in the movements of the masses of mountains. The Alpine "nappes" are a succession of large overthrusts, starting in earlier geologic epochs and lasting since then over long periods of time. Old crystalline rocks were found floating on younger layers. Separated from their roots, the strata in the upper portions of high mountains were described as resting over young sedimentary rocks, thus separated by what might, in our engineering language, now be termed as gigantic surfaces of slip. Overriding these gently inclined planes of thrust, whole mountains have been displaced by miles of distances. In the hands of the geologist these surfaces have served to decipher the riddles in the formation of high mountain chains just as their much smaller relatives have helped us today to describe more precisely some few of the plastic states of stress in engineering materials. They might in the future still serve to study by mechanical means and to reconstruct more precisely the history of the crustal movements of whole continents, which, since Alfred Wegener and F. B. Taylor, we know are nothing but thin shells drifting slowly and slipping over the earth's globe. It will be deeply regretted that progress was interrupted in this highly interesting new field of research by the untimely death in November, 1930, of Alfred Wegener, the founder of the theory of continental drift, while on his last expedition to Greenland, which was partly devoted to gathering further conclusive data about the comparatively rapid lateral drift and rise of this ice-covered continental plate. The upper solid strata respond to the lateral movements of the continental shells along these surfaces of slip, first probably produced at greater depths, but perhaps reaching in some places to the surface of the earth or the bottom of the ocean grounds, and later after the uplifting of the mountain chains shifted into their present positions. It is therefore highly probable that the laws of their formation will be not essentially different from those found by engineering laboratory tests and that from their further study the exact amounts of plastic compression of the continental plates at greater depth might be deduced. These plates will, in their upper portions behave as solid rocks do, at depths of some tens of miles and for short shocks or disturbances (earthquakes) they will have to be considered as rigid, but when exposed to geological actions of long duration and under the elevated temperatures prevalent at such depths they will yield plastically just as our main engineering materials do under similar conditions.

## CONCLUSIONS

Reviewing the preceding series of examples, by means of which the surfaces of slip or the plastic layers were either made visible in actual cases or could be constructed by mathematical methods, and recalling the number of cases in which the flow lines in the experiments and the lines predicted by analysis were found to agree fairly well in their shape with each other, we might draw the conclusion from these facts that observing these surfaces of slip in loose or in plastic materials might prove as helpful means to learn more about the nature of the various states of stress encountered in engineering. In the foregoing only a few names were mentioned; it is perhaps scarcely necessary to add that full acknowledgments are due to all those whose names, for the sake of brevity, were not mentioned, but whose attempts have materially helped to bring out the progress in this special field of investigation.

While the investigators of former times found themselves confronted with almost insurmountable difficulties and little hope seemed to remain that it may become possible to obtain more exact laws expressing the facts about plastic flow, the general trend of these developments seems to indicate that the time might perhaps not be so distant when a better order in the discussion of the mechanical properties of solids and of their various modes of deforming permanently will become more distinctly perceptible. Perhaps the surfaces of slip appearing on plastically deformed specimens will prove as helpful kinematic means to visualize the way in which they deform, comparable in their use with those early speculations in mathematical physics from which later important theories such as, for example, those of the potential and the potential functions developed, and which once originated in Faraday's simple conceptions of the lines of magnetic force.

## ACKNOWLEDGMENTS

Acknowledgments are due to the Westinghouse Electric and Manufacturing Co. for having put at the disposal of the speaker all facilities in the Research Laboratories in East Pittsburgh, Pa., for these investigations, and in particular to Messrs. L. W. Chubb, Director of the Laboratories, and J. M. Lessells, former Manager of the Mechanics Division for the most valuable help received at the laboratories. Acknowledgments are also due to Messrs. T. D. Lynch, Past-President of the American Society for Testing Materials, and to C. L. Warwick, Secretary-Treasurer of this Society, for their cooperation. The speaker wishes particularly to thank Messrs. C. W. MacGregor, W. O. Richmond, and P. M. Hengstenberg for their valuable assistance in preparing the various apparatus for the tests.

# SOME PHYSICAL PROPERTIES OF HARDENED TOOL STEEL

By J. V. Emmons[1]

## Synopsis

This paper reviews previous investigations on the mechanical properties of hardened tool steel. The utilization of various methods of testing for measuring such properties is also discussed.

A method of measuring the properties of strength and plasticity in hardened steels by means of a torsion test has been developed and is described in detail. From the measured properties of strength and plasticity, a value known as a coefficient of toughness is calculated.

By use of this method, a study is made of the properties of a typical tool steel over a wide range of heat treatments. The Rockwell hardness is determined and the microstructure recorded for the purpose of comparison. An analysis is made of these results upon the typical steel and the several properties are correlated. The bearing of the results obtained upon an existing theory of hardness is pointed out.

Conclusions are drawn as to the nature of the typical tool steel investigated, the theories in regard to the decomposition products of austenite and their properties and upon the general utility of the method.

This paper is part of a study being made of the fundamental properties which can affect the usefulness of tool steels. It early became evident that a knowledge of the physical properties of strength and toughness would be of great value. The problem has been to devise convenient means of making quantitative measurements. The property of hardness has received considerable attention from others and useful means are already available for determining it. Data as to the strength and toughness of tool steels are comparatively rare. A quantitative knowledge of these three properties, if correlated with the microstructure, should throw light upon some of the unexplained behavior of hardened steel. It was also hoped that from this study there could be devised means of evaluating the probable usefulness of new compositions of steel. This would avoid the slow, laborious and costly process of making complete series of cutting tests at a wide range of heat treatments.

A study was made of such previous work upon the physical properties of hardened tool steels or closely related steels as was available.

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[1] Metallurgist, The Cleveland Twist Drill Co., Cleveland, Ohio.

Tension and transverse bending tests were investigated by Mr. A. H. d'Arcambal(1)[1] who recorded results for the ultimate strength of carbon and high-speed steel but was not able to detect any deformation or ductility.

Bending and impact tests were investigated by Mr. R. K. Barry(2, 3) in an attempt to evaluate the toughness of high-speed steel. Barry considers that the total deflection in the bending test is a measure of the toughness.

Tensile and fatigue strength have been determined for a 1-percent carbon steel in the hardened and drawn condition by Mr. J. M. Lessells(4).

A torsion test was used by Albert Sauveur(5, 10) for determining the ductility or plasticity of various steels at elevated temperatures.

Tension and impact tests upon tool steels were discussed by Mr. J. P. Gill and Mr. L. D. Bowman(6) who consider these methods useful only in connection with drawing temperatures in excess of 650° F.

Methods of determining the torsional modulus of elasticity are discussed by Mr. W. P. Wood(7) who records results for some hardened and drawn steels commonly used for springs.

A very comprehensive summary of testing methods and bibliography are contained in the Symposium on the Effect of Temperature on Metals held in 1924(8

Tension and impact tests were used by Mr. A. R. Page and Mr. J. H. Partridge(9) to determine some properties of several valve steels containing chromium.

The torsional moduli of several spring materials were determined at varying temperatures by Messrs. F. P. Zimmerli, W. P. Wood and G. D. Wilson(12).

The form of torsion test specimens of the hollow cylindrical type was discussed by Messrs. R. L. Templin and R. L. Moore(13).

None of the above workers utilized their methods for a comprehensive study of the properties of tool steel. The torsion test does not appear to have been used at all for a study of the properties of tool steels or similar hard materials of a low degree of plasticity.

Preliminary trials of the above methods indicated that the tension test would give but little information as to the amount of deformation, which in steels of the type under consideration is extremely small. The flexure test was somewhat superior to the tension test, but due to the concentration of stresses, yielded erratic results. It should, however, be noted that for some special cases such as wires of small diameters, the flexure test may be made to yield useful information.

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[1] The boldface numbers in parentheses refer to the reports and papers given in the list of references appended to this paper, see p. 75.

## DESCRIPTION OF METHOD OF TESTING

A study was then made of the torsion test which seemed to offer the possibility of accurate measurement of very small amounts of deformation. After several trials a standard form of test specimen was adopted as shown in Fig. 1. Some of the advantages of this type of test specimen are, first, sufficient length to make the deformation of such magnitude as to be easily measured; second, avoidance of concentration of stresses; third, ease of heat treatment; fourth, low cost of manufacture, making possible economical production of test specimens in quantities.

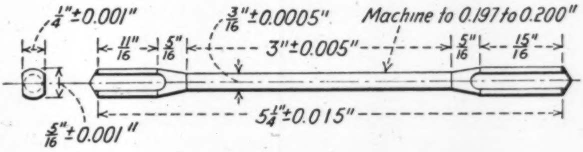

FIG. 1.—Torsion Test Specimen.

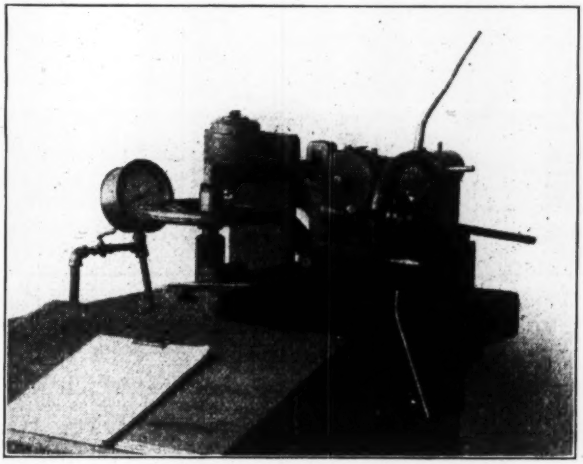

FIG. 2.—Showing the Torsion Testing Apparatus.

The machine used was an adaptation of an old machine originally built by the Cleveland Twist Drill Co. for measuring the torsional strength of small sized drills. This machine is shown in Fig. 2.

In performing a test, the torque is applied by means of the hand levers through gearing to the head chuck which holds the test specimen. The stress in the specimen is measured by conducting the torque from the tail chuck through levers to a rotating piston and a hydraulic gage. The torque is recorded in pounds operating at a 1-in. radius. The head chuck is direct-connected to a dial graduated in degrees of circular measure so that its rotation may be recorded with an accuracy of 1 deg. The deformation of the ends of the test specimens in the chucks is so slight that no appreciable error is thereby introduced in the values for the angle of twist. In ordinary practice, for each increment of 25 lb. in torque applied, the revolution of the head chuck in degrees is recorded. From these results, stress-strain curves may be plotted. Two operators are required, one of whom applies the torque and calls off each 25-lb. increase. The second operator records the angular deformation for each 25-lb. increment. The first operator notes the ultimate torque at which rupture occurs and the second operator, the ultimate angle of twist. When desired, the observations can be recorded by regular increments of twist instead of increments of torque. When the test specimen breaks, the fracture is immediately examined for evidences of seams or segregations which might have contributed to premature failure. If such are found, the results from this particular specimen are rejected and not included in the averages. There is apparently a nice balance between the tendency for the initial failure of the test specimens to start in tension or in shear. Some fail one way and some the other for no apparent cause. The specimens which fail in tension sometimes fly to pieces with great violence. It has been necessary to provide a "bullet proof" enclosure for the specimen, to prevent injury to the operators from flying fragments. A typical record sheet is

Table I.—Typical Record Sheet of Torsion Test Observations.

| Torque, in-lb. | Angle of Twist, deg. | | | |
|---|---|---|---|---|
| | Specimen No. 252 | Specimen No. 253 | Specimen No. 254 | Average |
| 50 | 12 | 13 | 10 | 12 |
| 75 | 16 | 19 | 15 | 17 |
| 100 | 21 | 21 | 18 | 20 |
| 125 | 25 | 24 | 22 | 24 |
| 150 | 30 | 30 | 27 | 29 |
| 175 | 34 | 34 | 31 | 33 |
| 200 | 40 | 39 | 36 | 38 |
| 225 | 45 | 45 | 42 | 44 |
| 250 | 52 | 52 | 48 | 51 |
| 275 | 63 | 65 | 60 | 63 |
| 300 | 89 | 92 | 75 | 85 |
| 325 | 133 | 140 | 130 | 134 |
| Ultimate angle, deg | 283 | 243 | 273 | 266 |
| Ultimate torque, in-lb | 335 | 350 | 342 | 342 |

shown in Table I, being the observations made upon the three test specimens hardened at 1650° F. (900° C.) and drawn at 450° F. (230° C.) as later described. These observations were used to plot the typical stress-strain curve shown in Fig. 3. In order to show the amount of scatter of the observations on this curve, each divergent value is plotted separately, the curve being drawn to the average.

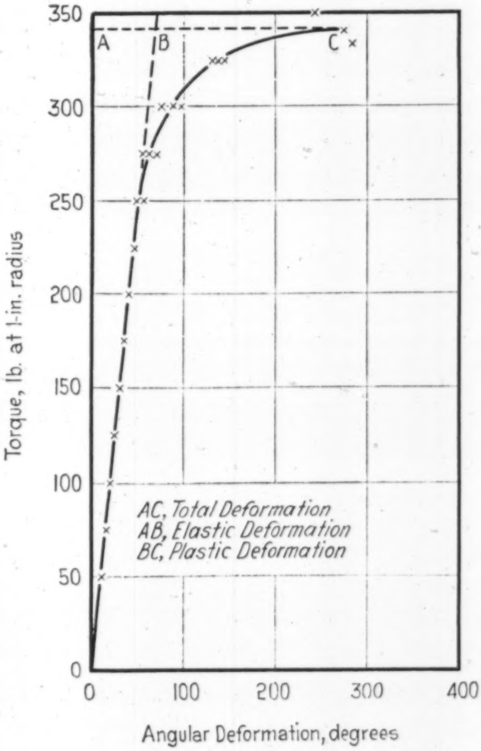

FIG. 3.—Typical Stress Strain Curve, Showing Scatter of Determinations and Method of Obtaining the Plastic Deformation.

Preliminary experiments indicated that an average of results of three test specimens which had been made from the same bar of steel and given the same heat treatment could be expected to be reliable. If the three specimens checked well, confidence was felt in the results. If the observations on the three showed a wide variation, the presence of uncontrolled variables was indicated. It was noted that there was normally a much wider variation in the observations of the total deformation than in those of the ultimate torque or of the hardness.

## THE PHYSICAL PROPERTIES DETERMINED

The ultimate torsional strength or modulus of rupture in torsion in the extreme fibers is not usually computed because of the errors which are introduced in the torsion formula by the variations in the plastic deformation. The ultimate torque is used as an indication of the strength of the specimen.

TABLE II.—OBSERVATIONS OF ULTIMATE TORQUE, IN-LB.

| Hardening Temperatures | Drawing Temperatures | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 212° F. (100° C.) | 350° F. (175° C.) | 400° F. (205° C.) | 450° F. (230° C.) | 500° F. (260° C.) | 550° F. (290° C.) | 600° F. (315° C.) | 650° F. (345° C.) | 700° F. (370° C.) | 750° F. (400° C.) |
| 1450° F. (790° C.). | 165 | 155 | 150 | 150 | 170 | 170 | 155 | 165 | 195 | 170 |
| 1550° F. (845° C.). | 347 | 335 | 325 | 320 | 315 | 305 | 290 | 265 | 260 | 240 |
| 1650° F. (900° C.). | 270 | 315 | 330 | 342 | 320 | 320 | 300 | 285 | 270 | 250 |
| 1750° F. (955° C.). | 205 | 280 | 300 | 305 | 330 | 306 | 273 | 265 | 240 | 205 |
| 1850° F. (1010° C.) | 145 | 200 | 235 | 245 | 287 | 280 | 285 | 260 | 225 | 225 |

TABLE III.—OBSERVATIONS OF TOTAL ANGULAR DEFORMATION, DEG.

| Hardening Temperatures | Drawing Temperatures | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 212° F. (100° C.) | 350° F. (175° C.) | 400° F. (205° C.) | 450° F. (230° C.) | 500°.F (260° C.) | 550° F. (290° C.) | 600° F. (315° C.) | 650° F. (345° C.) | 700° F. (370° C.) | 750° F. (400° C.) |
| 1450° F. (790° C.). | 49 | 90 | 155 | 151 | 137 | 66 | 174 | 87 | 56 | 192 |
| 1550° F. (845° C.). | 136 | 215 | 178 | 137 | 92 | 109 | 125 | 141 | 117 | 166 |
| 1650° F. (900° C.). | 111 | 234 | 316 | 260 | 123 | 145 | 139 | 198 | 143 | 152 |
| 1750° F. (955° C.). | 67 | 147 | 237 | 446 | 289 | 152 | 93 | 189 | 62 | 39 |
| 1850° F. (1010° C.) | 40 | 66 | 102 | 99 | 157 | 166 | 229 | 123 | 56 | 77 |

TABLE IV.—CALCULATED PLASTIC ANGULAR DEFORMATION, DEG.

| Hardening Temperatures | Drawing Temperatures | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 212° F. (100° C.) | 350° F. (175° C.) | 400° F. (205° C.) | 450° F. (230° C.) | 500° F. (260° C.) | 550° F. (290° C.) | 600° F. (315° C.) | 650° F. (345° C.) | 700° F. (370° C.) | 750° F. (400° C.) |
| 1450° F. (788° C.). | 21 | 64 | 130 | 126 | 109 | 37 | 148 | 59 | 22 | 164 |
| 1550° F. (843° C.). | 71 | 153 | 119 | 78 | 34 | 53 | 72 | 93 | 70 | 124 |
| 1650° F. (899° C.). | 63 | 176 | 255 | 197 | 64 | 86 | 84 | 146 | 94 | 107 |
| 1750° F. (954° C.). | 31 | 96 | 182 | 390 | 228 | 96 | 43 | 141 | 20 | 3 |
| 1850° F. (1010° C.) | 16 | 31 | 59 | 51 | 105 | 115 | 177 | 76 | 16 | 37 |

The total deformation may be divided into the elastic deformation which is proportional to the stress and the plastic deformation which occurs only at stresses beyond the proportional limit. Values for the plastic deformation are obtained in the following manner, again referring to Fig. 3. A typical stress-strain curve is plotted, usually from the group of test specimens that show the highest ultimate torque. The straight portion of the curve is then projected to the top of the chart. This straight line is regarded as representing the elastic deformation of the specimen. The value for the plastic deformation is then the total deformation at the ultimate torque minus the elastic deformation corresponding to the same torque. In the figure, the distance *AC* represents the total deformation, *AB* the elastic deformation and *BC* the plastic deformation.

From the ratio of the elastic deformation to the torque may be calculated the torsional modulus of elasticity or, as it is sometimes called, the modulus of rigidity.

The plastic deformation is one of the fundamental properties of the steel which has a large effect upon the toughness and is sometimes confused with it. As has been pointed out by Jeffries and Archer(11) a true conception of the property of toughness involves the idea of strength as well as that of deformation.

Any attempt to evaluate toughness numerically involves combining strength with deformation in some proportion. Both elastic and plastic deformation without doubt contribute to toughness so that the total deformation must be used. For the particular test specimen and method here used, it was found that the product of the ultimate torque in pounds at 1-in. radius and the total angular deformation in degrees, combined the strength and deformation in about equal proportions for the case of very strong and tough steels. A number so obtained was believed to give at least a rough approximation of the toughness and has been termed a "coefficient of toughness."

## PREPARATIONS FOR AN EXPERIMENT

In preparing for an experiment, care is taken to provide sufficient steel which is identical as to composition, method of manufacture and structure to supply the desired number of test specimens. For this purpose it is convenient to use a coil of wire rod. Careful inspection is made of both ends of the coil by means of the microscope and hardness tests, to insure that the structure of the steel and the response to heat treatment are uniform. Every effort is made to insure that all test specimens are machined in a uniform manner and are free from seams or other defects. The same care is used to secure uniformity of heat treatment, especial precautions being taken to control the temperatures, the soaking times and the method of cooling. It is especially desired that the test specimens come through the heat treatment with a minimum of distortion as no straightening is permitted before testing.

## APPLICATION OF METHOD

This testing method was applied to a chromium tool steel of the following analysis:

| | |
|---|---|
| Carbon, per cent | 1.12 |
| Manganese, per cent | 0.19 |
| Phosphorus, per cent | 0.014 |
| Silicon, per cent | 0.30 |
| Sulfur, per cent | 0.007 |
| Chromium, per cent | 1.07 |

It was particularly desired to investigate the mechanical properties of this steel as developed by various hardening and drawing temperatures. It was shown by the author in 1912(14) that the method of distribution of the cementite has a great effect upon the properties of tool steel. In order to reduce the effect of such variations in the initial distribution to a minimum, great care was exercised to secure a coil of wire with a very uniform structure. See Fig. 10 of the annealed structure before hardening (× 2000). For purpose of comparison with the strength, the plasticity and the toughness as determined by the torsion test, the hardness was also determined by the Rockwell method.

A schedule of heat treatments was prepared to cover a large range of both hardening and drawing temperatures, including temperatures both below and above those commonly regarded as useful. All hardening was done from a lead bath. The quenching bath was oil. The soaking time at the hardening temperature was 60 seconds. The soaking time at the drawing temperature was 30 minutes. Each heat treatment was run in triplicate, requiring for the 50 variations, 150 test specimens, which were all from one coil of wire rod.

The complete schedule of heat treatments was as follows:

| Hardening Temperatures | Drawing Temperatures | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 212° F. (100° C.) | 350° F. (175° C.) | 400° F. (205° C.) | 450° F. (230° C.) | 500°.F (260° C.) | 550° F. (290° C.) | 600° F. (315° C.) | 650° F. (345° C.) | 700° F. (370° C.) | 750° F). (400° C. |
| 1450° F. (790° C.). | X | X | X | X | X | X | X | X | X | X |
| 1550° F. (845° C.). | X | X | X | X | X | X | X | X | X | X |
| 1650° F. (900° C.). | X | X | X | X | X | X | X | X | X | X |
| 1750° F. (955° C.). | X | X | X | X | X | X | X | X | X | X |
| 1850° F. (1010° C.) | X | X | X | X | X | X | X | X | X | X |

The 150 test specimens were then broken in the machine described above. The observations were tabulated as shown in the typical data sheet in Table I, which is a record of the results upon the three test specimens hardened at 1650° F. (900° C.) and drawn at 450° F.

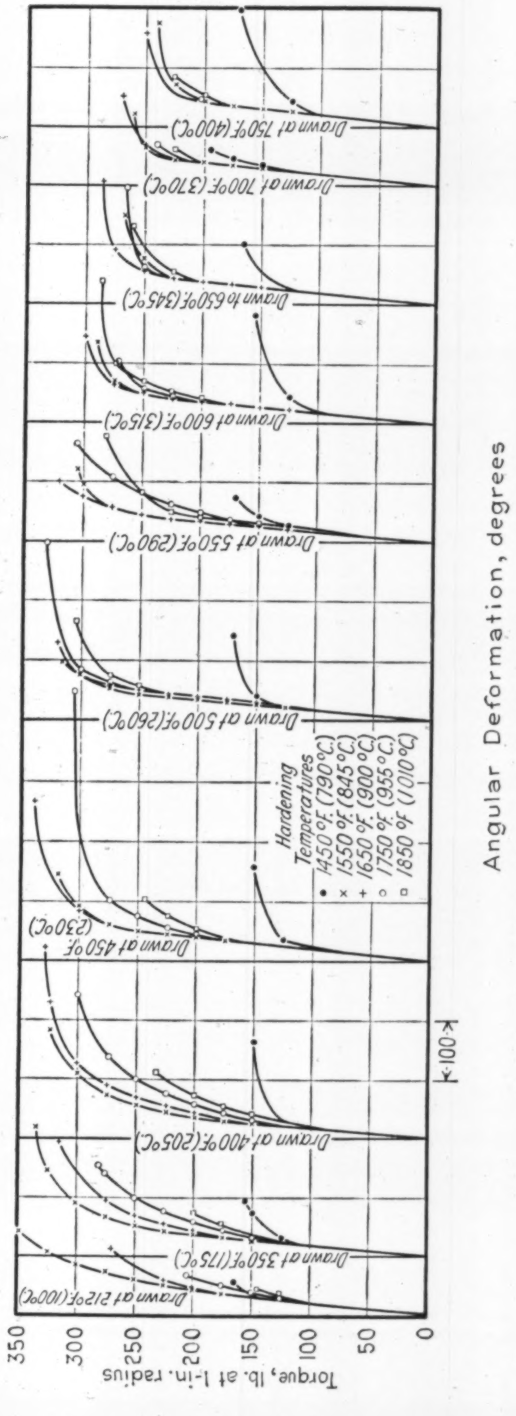

FIG. 4.—Stress-Strain Curves for Specimens Drawn at Various Temperatures.

(230° C.). Because of limitations of space, the remainder of the detailed observation sheets are not shown. From these detailed observations, stress strain curves were plotted as shown in Fig. 4.

Preliminary consideration of these curves indicates that the variations in heat treatment have developed very pronounced differences in physical properties. Both the ultimate torque and the total deformation show changes through a wide range of values and appear to have responded systematically to the variations in heat treatment.

TABLE V—CALCULATED COEFFICIENTS OF TOUGHNESS.

| Hardening Temperatures | Drawing Temperatures | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 212° F. (100° C.) | 350° F. (175° C.) | 400° F. (205° C.) | 450° F. (230° C.) | 500°.F (260° C.) | 550° F. (290° C.) | 600° F. (315° C.) | 650° F. (345° C.) | 700° F. (370° C.) | 750° F. (400° C.) |
| 1450° F. (790° C.) | 8 000 | 14 000 | 23 000 | 23 000 | 23 000 | 11 000 | 27 000 | 14 000 | 11 000 | 33 000 |
| 1550° F. (845° C.) | 47 000 | 72 000 | 58 000 | 44 000 | 29 000 | 33 000 | 36 000 | 37 000 | 30 000 | 40 000 |
| 1650° F. (900° C.) | 30 000 | 74 000 | 104 000 | 88 000 | 39 000 | 46 000 | 42 000 | 56 000 | 39 000 | 38 000 |
| 1750° F. (955° C.) | 14 000 | 41 000 | 71 000 | 136 000 | 95 000 | 47 000 | 25 000 | 50 000 | 15 000 | 8 000 |
| 1850° F. (1010° C.) | 6 000 | 13 000 | 24 000 | 24 000 | 45 000 | 46 000 | 65 000 | 32 000 | 13 000 | 17 000 |

TABLE VI.—OBSERVATIONS OF ROCKWELL HARDNESS ("C" SCALE).

| Hardening Temperatures | Drawing Temperatures | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 212° F. (100° C.) | 350° F. (175° C.) | 400° F. (205° C.) | 450° F. (230° C.) | 500° F. (260° C.) | 550° F. (290° C.) | 600° F. (315° C.) | 650° F. (345° C.) | 700° F. (370° C.) | 750° F. (400° C.) |
| 1450° F. (790° C.) | 59.3 | 57.6 | 56.2 | 55.8 | 54.7 | 52.8 | 52.3 | 50.4 | 49.0 | 45.3 |
| 1550° F. (845° C.) | 66.5 | 62.9 | 62.2 | 59.8 | 58.2 | 57.2 | 55.4 | 53.3 | 52.3 | 50.2 |
| 1650° F. (900° C.) | 67.1 | 65.6 | 63.6 | 61.4 | 60.0 | 59.0 | 57.3 | 55.4 | 53.4 | 51.6 |
| 1750° F. (955° C.) | 63.1 | 63.5 | 63.3 | 62.5 | 59.7 | 58.6 | 57.5 | 55.5 | 54.0 | 52.8 |
| 1850° F. (1010° C.) | 61.8 | 62.1 | 60.9 | 60.4 | 59.3 | 57.7 | 56.1 | 54.4 | 53.3 | 51.7 |

Table II shows the observations of the ultimate torque. Table III shows the observations of the total deformation or angle of twist. Table IV shows the calculated angles of plastic deformation. Table V shows the calculated coefficients of toughness, the numbers being reduced to the nearest thousand for simplicity. Further analysis of the observations is made by plotting the values for ultimate torque against drawing temperatures as shown in Fig. 5, also the values for plastic deformation against drawing temperatures as shown in Fig. 6. The coefficients of toughness are plotted against variable drawing temperatures in Fig. 7.

## HARDNESS DETERMINATIONS

In order to throw as much light as possible upon the properties of the steel under consideration, determinations were also made of the Rockwell hardness using the diamond cone and the "C" scale. The

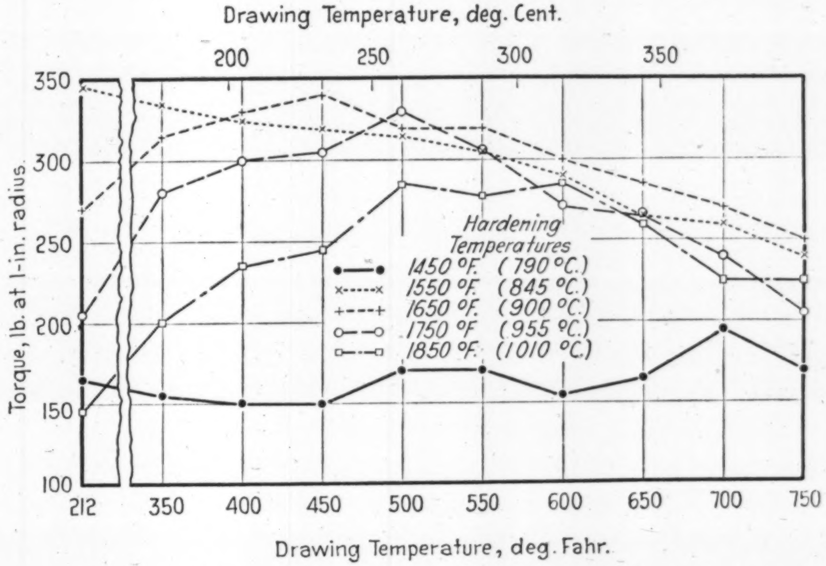

FIG. 5.—Ultimate Torque at Varying Drawing Temperatures.

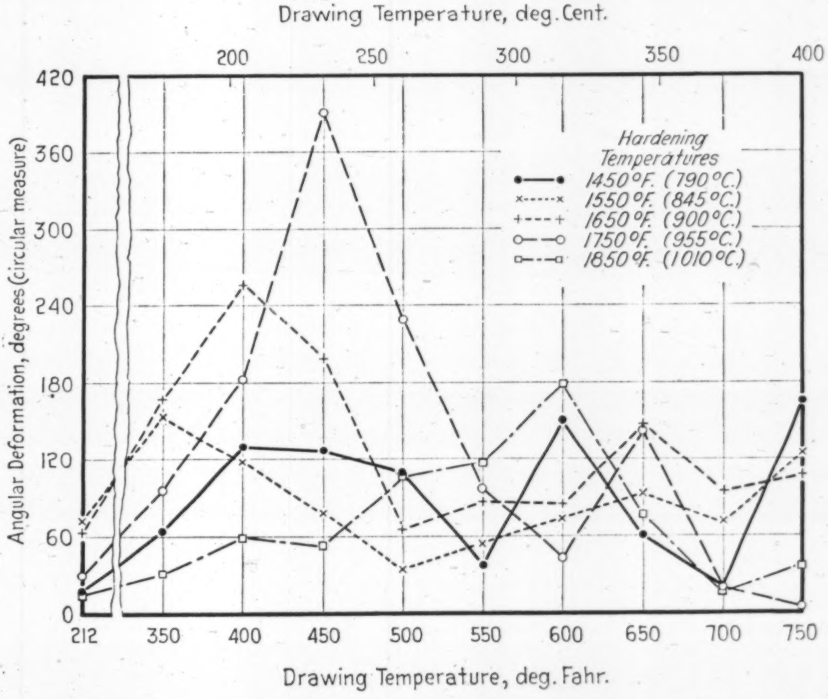

FIG. 6.—Plastic Deformation at Varying Drawing Temperatures.

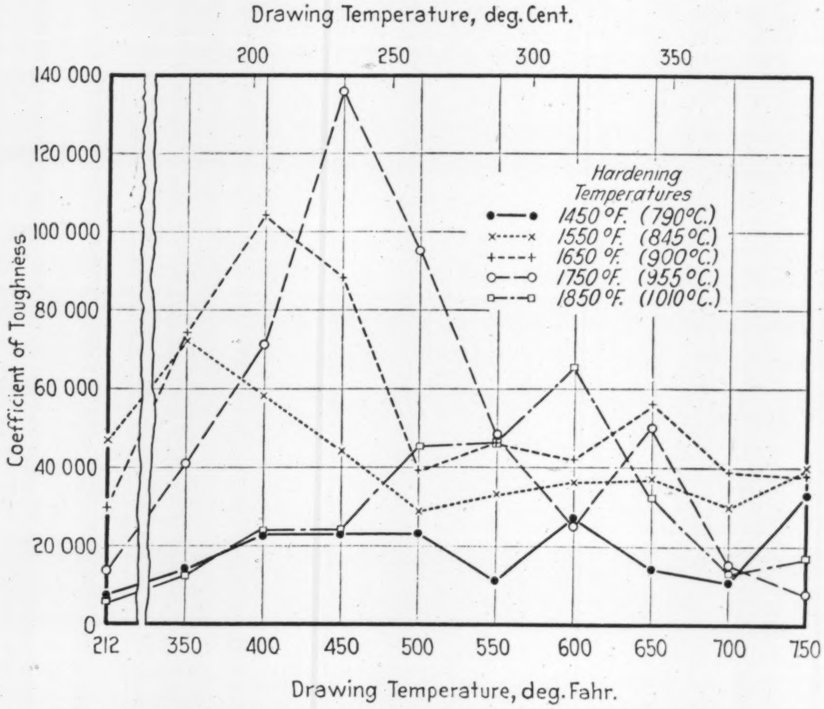

FIG. 7.—Coefficient of Toughness at Varying Drawing Temperatures.

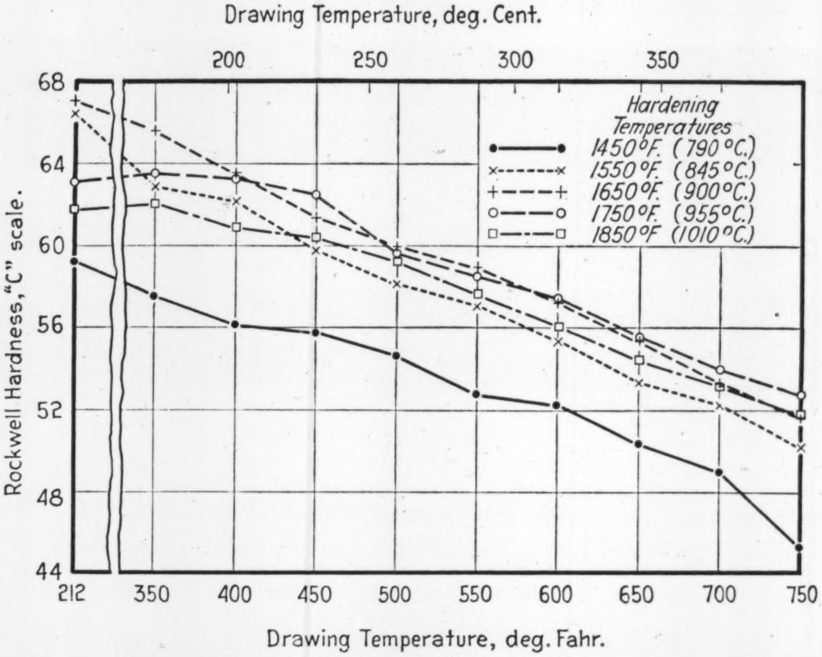

FIG. 8.—Rockwell Hardness ("C" scale) at Varying Drawing Temperatures.

results of these determinations, as shown in Table VI, are averaged from three tests upon each of the three pieces at each heat treatment. Curves were plotted for Rockwell hardness against drawing temperatures as shown in Fig. 8. In order to show more clearly the effect of the variations of the heat treatment upon the hardness, curves were also plotted for Rockwell hardness against hardening temperatures as shown in Fig. 9.

## MICROSTRUCTURE

To provide a basis for comparison with the mechanical properties, the microstructures of specimens at all heat treatments were examined. Typical structures were photographed as shown by the following list. All are at magnifications of 2000 diameters.

| | |
|---|---|
| Annealed structure before hardening | Fig. 10 |
| Hardened at 1450° F. (790° C.) and drawn at 212° F. (100° C.) | Fig. 11 |
| Hardened at 1550° F. (845° C.) and drawn at 212° F. (100° C.) | Fig. 12 |
| Hardened at 1650° F. (900° C.) and drawn at 212° F (100° C.) | Fig. 13 |
| Hardened at 1750° F. (955° C.) and drawn at 212° F. (100° C.) | Fig. 14 |
| Hardened at 1850° F. (1010° C.) and drawn at 212° F. (100° C.) | Fig. 15 |
| Hardened at 1450° F. (790° C.) and drawn at 750° F. (400° C.) | Fig. 16 |
| Hardened at 1650° F. (900° C.) and drawn at 450° F. (230° C.) | Fig. 17 |
| Hardened at 1650° F. (900° C.) and drawn at 750° F. (400° C.) | Fig. 18 |
| Hardened at 1850° F. (1010° C.) and drawn at 500° F. (260° C.) | Fig. 19 |
| Hardened at 1850° F. (1010° C.) and drawn at 600° F. (315° C.) | Fig. 20 |
| Hardened at 1850° F. (1010° C.) and drawn at 750° F. (400° C.) | Fig. 21 |

## ANALYSIS OF THE PROPERTIES OF THE STEEL

First considering the stress-strain curves in Fig. 4, it is obvious that the variations in heat treatment have greatly affected the strength, the elastic and plastic deformation. The plastic deformation begins almost imperceptibly with no definite yield point. The proportional limit is so difficult to locate precisely in torsion, that the attempt was not made in this experiment.

The modulus of elasticity in torsion was calculated to be 9,400,000 lb. per sq. in. In these calculations, suitable corrections were introduced for the elastic deformation of the tapered ends of the specimens. As the test specimen here used is not designed for the purpose of determining the modulus of elasticity, no particular accuracy is claimed for this value. This modulus did not appear to have been affected by the variations in heat treatment used in this experiment.

The maximum strength observed was in the specimens hardened at 1550° F. (845° C.) and drawn at 212° F. (100° C.). As these specimens showed low plastic deformation, a calculation of the ultimate torsional strength in the extreme fibers would be nearly correct. The value obtained was 268,000 lb. per sq. in.

The ultimate torque, as shown in Fig. 5, is greatly affected by variations in the drawing temperatures. Each hardening temperature also produces a curve which has its own individuality. For the specimens hardened at 1450° F. (790° C.) the strength is of a low order and is but little changed by variations in the draw. For the specimens hardened at 1550° F. (845° C.) the strength is at a maximum at the 212° F. (100° C.) draw and decreases quite constantly with increasing drawing temperatures. For the specimens at the higher hardening temperatures, the strength has a low value at the 212° F. (100° C.) draw and then rises to a peak, after passing which, it falls off in a similar manner to the curve for 1550° F. (845° C.). The location of the peaks of strength are as follows:

For 1550° F. (845° C.) at a draw of 212° F (100° C.)
For 1650° F. (900° C.) at a draw of 450° F. (230° C.)
For 1750° F. (955° C.) at a draw of 500° F. (260° C.)
For 1850° F. (1010° C.) at a draw of 500° F. (260° C.) and 600° F. (315° C.)

This shows that increasing the hardening temperature increases the drawing temperature at which the peak of strength occurs. For drawing temperatures above 350° F. (175° C.), the hardening temperature of 1650° F. (900° C.) appears to produce the highest average strength.

The plastic deformation as shown in Fig. 6, likewise displays a characteristic curve in which the values start low at 212° F. (100° C.), rise to a peak at low drawing temperatures, drop violently to low points between the draws of 500° F. (260° C.) and 700° F. (370° C.), then start an irregular recovery. If we eliminate the results for the specimens hardened at 1450° F. (790° C.) which are known to be only partially hardened, the locations of the peaks and subsequent valleys are controlled in a definite way by the hardening temperatures as follows:

| HARDENING TEMPERATURE | PEAK OF PLASTIC DEFORMATION | SUBSEQUENT LOW VALUE |
|---|---|---|
| 1550° F. (845° C.)........ | 350° F. (175° C.) | 500° F. (260° C.) |
| 1650° F. (900° C.)........ | 400° F. (205° C.) | 500° F. (260° C.) |
| 1750° F. (955° C.)........ | 450° F. (230° C.) | 600° F. (315° C.) |
| 1850° F. (1010° C.)........ | 600° F. (315° C.) | 700° F. (370° C.) |

The height of some of the peaks is startling. The valleys follow the peaks at draws averaging 125° F. (70° C.) higher. These valleys bear a startling resemblance to so-called "blue brittleness." The drawing temperatures used were not sufficiently high to give much of a picture of the plastic recovery as the annealing range is approached.

The curves plotted for the coefficient of toughness in Fig. 7, being a combination of the results for the ultimate torque and the plastic deformation, show a similar shape. The highest value for toughness is at the hardening temperature of 1750° F. (955° C.) with a draw of 450° F. (230° C.). The peaks are in all cases sharp ones, indicating the necessity of accurate heat treatment if the maximum toughness is sought. The drop in toughness after the peak is passed

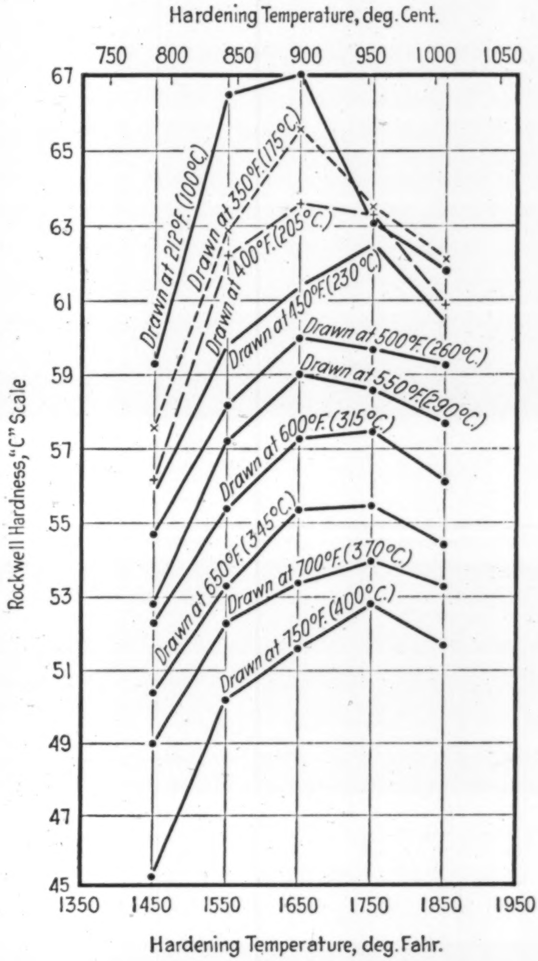

FIG. 9.—Rockwell Hardness ("C" scale) at Varying Hardening Temperatures.

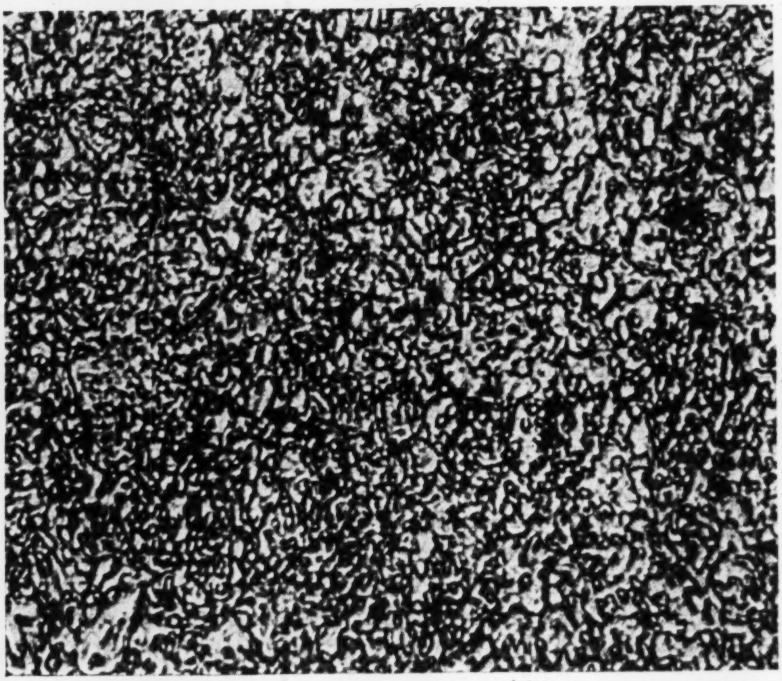

FIG. 10.—Annealed Structure Before Hardening (× 2000).

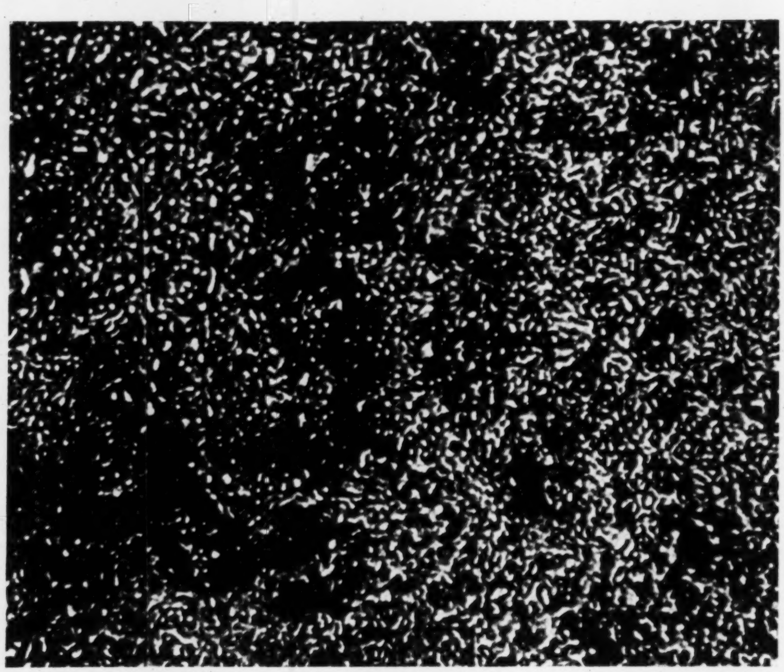

FIG. 11.—Structure Hardened at 1450° F. (788° C.) and Drawn at 212° F. (100° C.) (× 2000).

is sharp and positive. It is obvious that the toughness of this steel is a long way from being proportional to the drawing temperature as is sometimes assumed to be the case with hardened tools in general.

The curves plotted for the Rockwell hardness in Fig. 8 show in general a regular decrease in hardness with increasing drawing temperatures. There are two exceptions to this in the case of the specimens hardened at 1750° F. (945° C.) and 1850° F. (1010° C.). These specimens show a higher hardness at the 350° F. (175° C.) draw than at 212° F. (100° C.). In Fig. 9, the lower drawing temperatures are seen to have a peak hardness at a hardening temperature of 1650° F. (900° C.), while the higher drawing temperatures reach their hardness peak at 1750° F. (955° C.).

In comparing the curves for the several properties with each other, it is observed that the curves for ultimate torque, plastic deformation and Rockwell hardness have each their own characteristics which bear little relation to each other, except in occasional instances. Two of the interesting exceptions are as follows: First, for specimens hardened at 1850° F. (1010° C.), the curves for ultimate torque and plastic deformation are very similar in shape, both rising from low values to a peak at a draw of 600° F. (315° C.) then falling off again. This coincidence gives the specimens hardened at 1850° F. (1010° C.) and drawn at 600° F. (315° C.) a noteworthy degree of toughness for such a high hardening temperature. Second, for specimens hardened at 1550° F. (845° C.) the curves for ultimate torque and hardness are very similar, both showing a regular decrease in value for increasing drawing temperatures. For the higher hardening temperatures 1650° F. (900° C.), 1750° F. (955° C.), and 1850° F. (1010° C.), the strength and hardness are inversely proportional until after the peak of strength has been passed, when they become proportional. It is a noteworthy observation that the hardness and the toughness bear no simple relation to each other, particularly as it is so often assumed in tool hardening practice that hardness and toughness are inversely proportional.

## ANALYSIS OF MICROSTRUCTURE

The annealed structure in Fig. 10 shows very finely divided particles of spheroidized cementite in a matrix of ferrite. This is considered to show an excellent degree of refinement and a suitable condition for uniform hardening. Due to this uniform distribution of the cementite, the differences in the mechanical properties observed in this experiment are believed to be differences in the properties of the matrix except when otherwise noted.

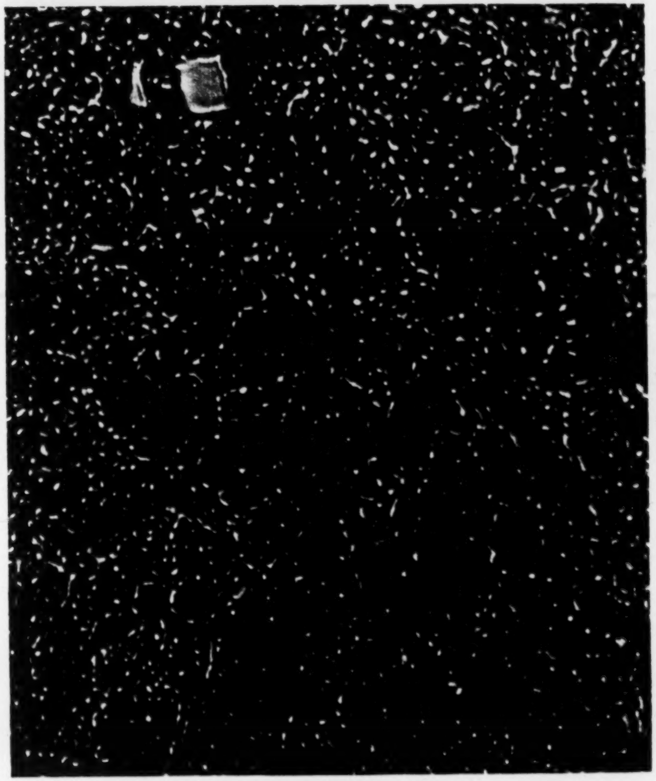

FIG. 12.—Structure Hardened at 1550° F. (843° C.) and Drawn at 212° F. (100° C.) (× 2000).

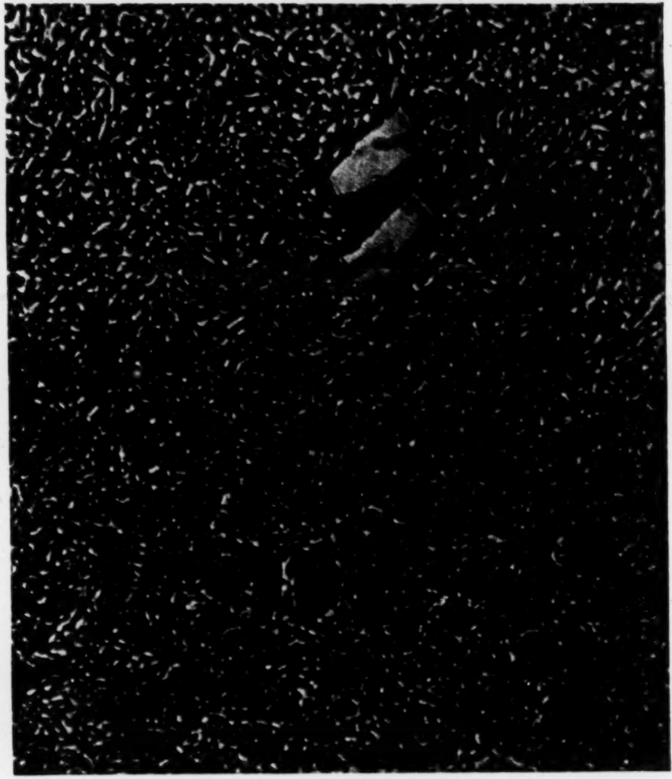

FIG. 13.—Structure Hardened at 1650° F. (899° C.) and Drawn at 212° F. (100° C.) (× 2000).

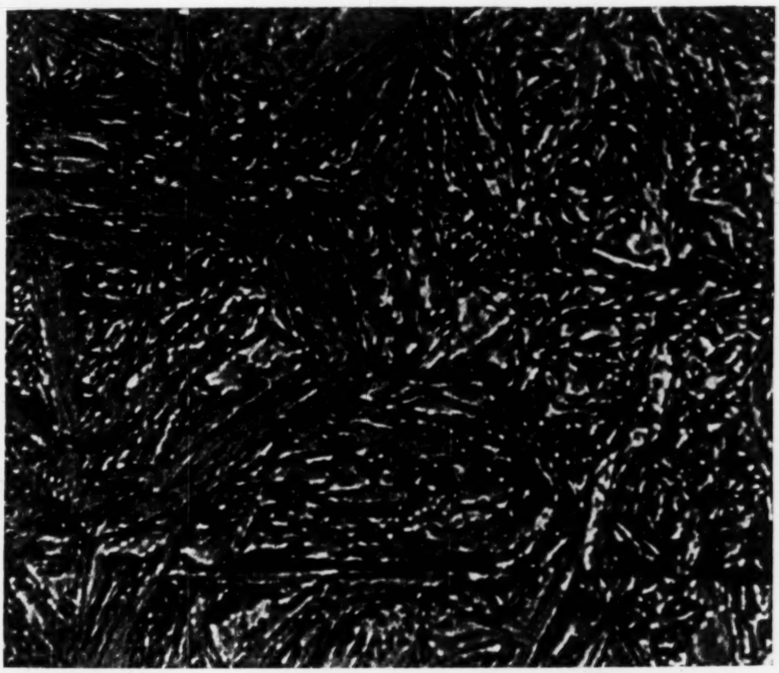

FIG. 15.—Structure Hardened at 1850° F. (1010° C.) and Drawn at 212° F. (100° C.) (× 2000).

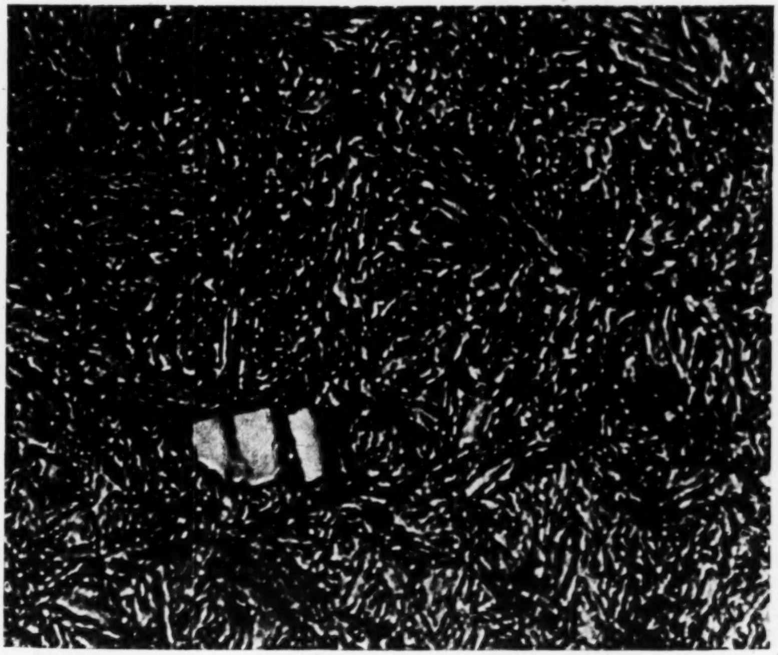

FIG. 14.—Structure Hardened at 1750° F. (954° C.) and Drawn at 212° F. (100° C.) (× 2000).

The structure of the specimen hardened at 1450° F. (790° C.) and drawn at 212° F. (100° C.) in Fig. 11 indicates clearly that only partial hardening has taken place. Comparatively few of the cementite particles have been taken into solution. This condition was already strongly suspected from the results obtained for the physical properties. The peculiar behavior of these specimens in the several tests is thus explained. It is surprising that a Rockwell hardness ("C" scale) as high as 59 was obtained with this structure. The low values for strength and plastic deformation are attributed partly to the presence of large quantities of undissolved massive cementite and partly to the lack of precipitated carbide particles to inhibit the grain growth of the ferrite in the troostite matrix.

The structure of the specimen hardened at 1550° F. (845° C.) and drawn at 212° F. (100° C.) is illustrated by Fig. 12. This specimen is fully hardened but there are still large numbers of very small cementite particles which have not yet gone into solution. Occasional large particles of cementite are scattered through the structure. This specimen has the maximum strength observed in this experiment, together with high hardness, but low plastic deformation. The high value for strength is attributed to the large proportion of martensite present and to its extremely fine crystallization. The low plastic deformation appears to be an inherent property of such finely crystallized martensite.

The structure of the specimen hardened at 1650° F. (900 ° C.) and drawn at 212° F. (100° C.) is shown in Fig. 13. This specimen has the maximum hardness observed in this experiment. The fine cementite particles have been largely taken into solution although many small ones and a few massive particles still persist. The strength is slightly below that of the structure shown in Fig. 12 but is still very high. The plastic deformation is practically unchanged. No definite traces of austenitic crystallization as yet appear in this structure. But little diffusion appears to have taken place so that the location of the cementite shows little or no change from that in the annealed structure.

In Fig. 14 is shown the structure of a specimen hardened at 1750° F. (955° C.) and drawn at 212° F. (100° C.). This structure shows the beginning of a definite austenitic crystallization. The small cementite particles are now nearly all in solution although a few massive particles still persist. The specimen hardened at this temperature is the first to show diffusion and recrystallization upon a scale large enough to be observed by the microscope. This beginning of aus-

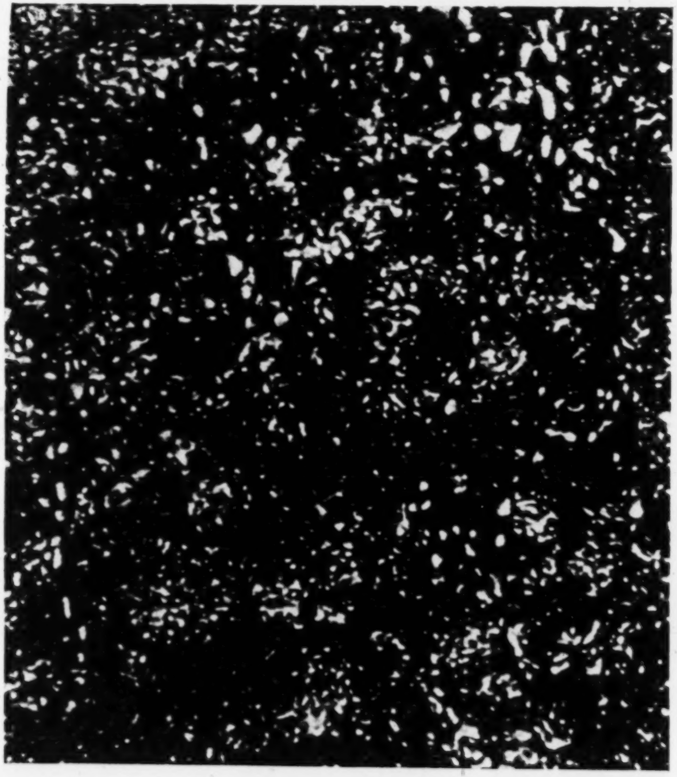

FIG. 16.—Structure Hardened at 1450° F. (788° C.) and Drawn at 750° F. (400° C.) (× 2000).

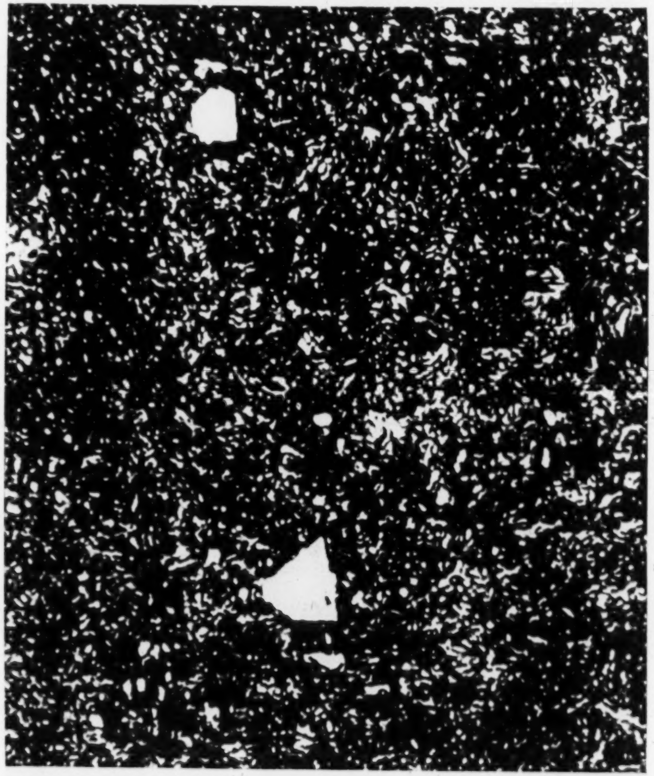

FIG. 17.—Structure Hardened at 1650° F. (899° C.) and Drawn at 450° F. (230° C.) (× 2000).

tenitic recrystallization has been accompanied by definite drops in the strength, the plastic deformation and the hardness.

In Fig. 15, appears the structure of a specimen hardened at 1850° F. (1010° C.) and drawn at 212° F. (100° C.). It is now predominantly austenitic in appearance and the crystalline grains are larger than in Fig. 14. Free cementite has now almost completely disappeared, and only in rare cases can a few massive particles be found. The hardness has dropped to Rockwell 61.8 ("C" scale), while the strength is the lowest observed in the entire experiment and the plastic deformation is almost at the vanishing point. The cause of these profound changes in the properties is none too apparent. The reduction of hardness in the face of the solution of an increased amount of cementite is believed to be caused by the retention of an increased amount of austenite. The comparatively small amount of reduction in the hardness indicates that the actual austenite present may be less than would appear from the structure. It is probable that the actual condition is that austenite crystalline grains have first formed at the 1850° F. (1010° C.) temperature; then during the quenching a large part of the austenite has altered to martensite, which has retained the crystal form of the austenite, forming what may be called martensite pseudomorphs after austenite. The presence of large amounts of martensite is further indicated by the fact that the specimen is still strongly magnetic. If this be true only a small rise in hardness would be expected when the retained austenite is converted to martensite by drawing. This is actually the case as is shown in Fig. 8 where drawing to 350° F. (175° C.) produces only a slight rise in the hardness of the two specimens observed to contain austenite. From the fact that this curve of hardness does not regain its normal slope until after the draw of 500° F. (260° C.) is passed, it is indicated that some austenite is still being converted to martensite at this temperature. This is confirmed by the microstructure in Fig. 19 as discussed later. The very low values for strength and plastic deformation may possibly be explained on the basis of the structure being chiefly martensite, entrapped in enormous crystalline grains of austenite and under the strained conditions of a mixed crystallization.

In Fig. 16 is found the structure of a specimen hardened at 1450° F. (790° C.) and drawn at 750° F. (400° C.). The effect of this high draw upon a partially hardened structure may be observed by comparing with the 212° F. (100° C.) draw in Fig. 11. The martensite is now converted to troostite but the cementite particles are still largely in the massive form in which they were observed in the annealed structure. The strength of this specimen is almost exactly

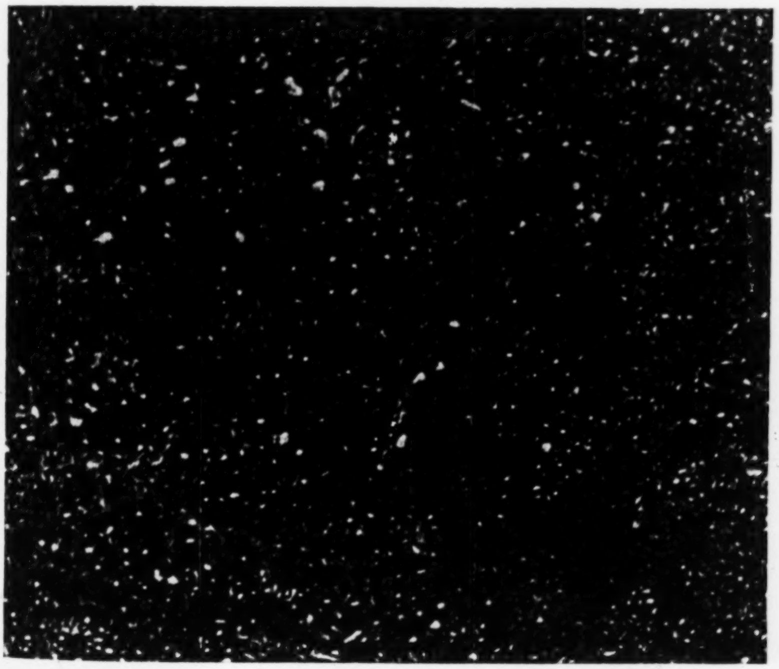

FIG. 18.—Structure Hardened at 1650° F. (899° C.) and Drawn at 750° F. (400° C.) (× 2000).

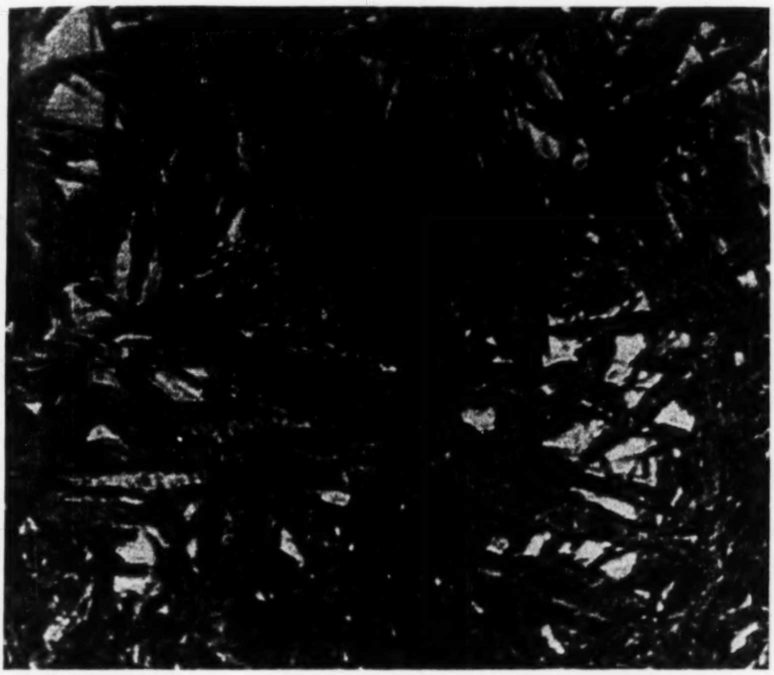

FIG. 19.—Structure Hardened at 1850° F. (1010° C.) and Drawn at 500° F. (260° C.) (× 2000).

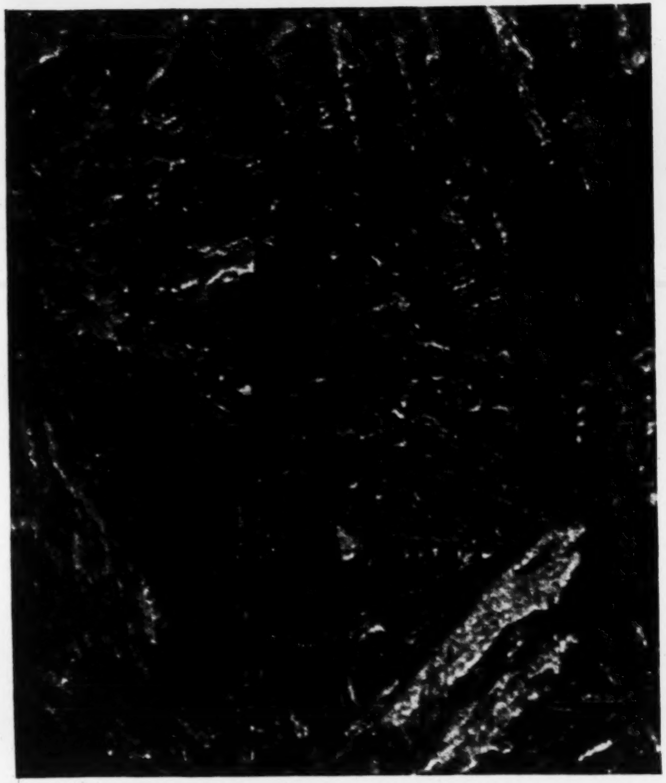

FIG. 20.—Structure Hardened at 1850° F. (1010° C.) and Drawn at 600° F. (315° C.) (× 2000).

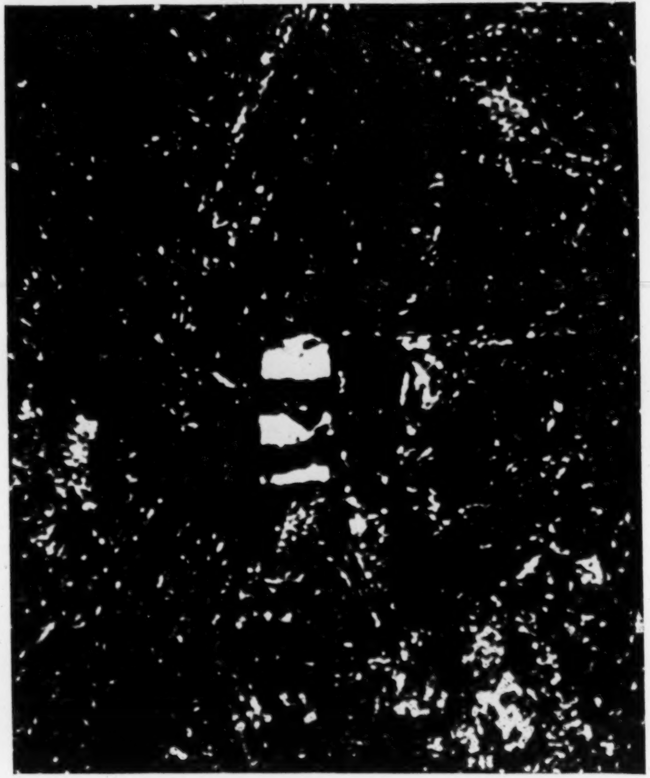

FIG. 21.—Structure Hardened at 1850° F. (1010° C.) and Drawn at 750° F. (400° C.) (× 2000).

the same as that of the specimen in Fig. 11. The plastic deformation has increased slightly while the hardness has greatly decreased.

The structure of the specimen hardened at 1650° F. (900° C.) and drawn at 450° F. (230° C.) is shown in Fig. 17. The effect of this drawing temperature can be examined by comparing with the 212° F. (100° C.) draw in Fig. 13. The martensite has now been partially converted to troostite. This structure is noteworthy as having produced the highest strength observed after any considerable amount of drawing. The plastic deformation is high but is materially below the peak for this hardening temperature which occurred at the 400° F. (205° C.) draw. The hardness has fallen to 61.4, Rockwell "C" scale.

Figure 18 shows the structure of a specimen at the same hardening treatment as in Figs. 13 and 17, but drawn to 750° F. (400° C.). The martensite is now completely converted into troostite which is in a fine, almost structureless condition. The strength has fallen off reluctantly with the higher draws and this structure is still the strongest of those which have been drawn at 750° F. (400° C.). The plastic deformation is low which can best be accounted for by the assumption that the precipitated cementite particles in the troostite have a size which produces great slip interference. The hardness has dropped to 51.6, Rockwell "C" scale, which is about the average for all of the specimens drawn at this temperature.

The structure of the specimen hardened at 1850° F. (1010° C.) and drawn at 500° F. (260° C.) in Fig. 19 shows the austenite largely but not entirely converted to martensite. It may be compared with the structure at the 212° F. (100° C.) draw in Fig. 15. While some conversion of the martensite to troostite has doubtless taken place, the martensite appears to be quite resistant to drawing. The strength is at the peak for this hardening temperature. This peak is here broad, extending from draw 500° F. (260° C.) to draw 600° F. (315° C.).

At the 600° F. (315° C.) draw, as shown in Fig. 20, the austenite has been entirely converted to martensite while the troostite is more prominent. Here we have the only case where the peaks for strength and plastic deformation coincide. This coincidence may be explained by the great heterogeneity of the structure. We have at the same time carbon-rich austenite decomposing to carbon-rich martensite and carbon-poor martensite decomposing to troostite; the resistance of austenite and martensite to decomposition appearing to be increased by the amount of carbon they contain. This coincidence of the peaks produces at this point a toughness which is unexpectedly great in view of the coarse crystallization. The hardness is about the average for this drawing temperature being 56.1, Rockwell "C" scale.

Figure 21 shows the structure of a specimen hardened at 1850° F. (1010° C.) and drawn at 750° F. (400° C.). The austenite and martensite have now practically disappeared but the outlines of the former austenite grains are still vaguely marked by the cementite particles, which have been precipitated from them. The extent to which the carbon has been concentrated in the large austenite flakes is now apparent. This structure is characterized by low hardness, low strength and low plasticity.

## GENERAL THEORY

If we attempt to interpret the above results in terms of the slip interference theory of Jeffries and Archer, the following conclusions are reached which are of course valid only for this particular steel. It should be recalled that the region of drawing temperatures below 212° F. (100° C.) is not under observation in this experiment. Such effects as might be due to crystallization in the body-centered tetragonal lattice have been largely dissipated by the 212° F. (100° C.) draw.

The Rockwell hardness reaches its maximum with the greatest amount of martensite present. The hardness of martensite also increases with increasing amounts of dissolved or entrapped carbon and with increasing refinement of grain size. When martensite alone constitutes the original matrix of the steel as quenched, increasing drawing temperatures produce a reduction in hardness that is readily explained by an increase in the ferrite grain size alone. When, due to high heating temperatures before quenching, austenite is retained, a drawing temperature of 350° F. (175° C.) produces a rise in hardness because the conversion of austenite to martensite is slightly greater in its effect than the increase in the ferrite grain size at this temperature.

The plastic deformation for martensite is low. If we assume that the cementite has not yet been completely precipitated from the solid solution by the 212° F. (100° C.) draw, this low plasticity may be explained by the strained condition present due to the alpha iron being highly supersaturated with carbon. The plasticity is still lower, in fact approaching zero, for the mixture of martensite and coarsely crystalline austenite produced by high hardening temperatures. When martensite is heated slightly so that atomic movement may begin, there is an immediate increase in the plasticity. This probably coincides with the formation of the smallest possible particles of iron carbide from the free iron and carbon atoms in the martensite and the accompanying release of strains. This particle size of the cementite must be definitely below the critical particle size which exerts the maximum keying effect on the crystallographic slip planes of the ferrite. The plastic deformation thus reaches its peak at an early stage in the drawing at the very beginning of the decompositon of the martensite. The plasticity is interfered with by too small a grain size of the martensite as well as too large. The most favorable grain size of the martensite for the production of maximum plasticity appeared to be developed by a hardening temperature of 1750° F. (955° C.). As the particles of the cementite grow to the critical size which exerts the maximum keying effect, the plasticity drops sharply to low values. It does not recover from these low values until the particle size becomes easily visible in the microscope with the formation of sorbite. This is beyond the range of drawing temperatures considered in this experiment.

The greatest observed strength was developed in nearly pure martensite containing less than the maximum amount of carbon in solution and having the smallest grain size. This particular structure, produced by hardening at 1550° F. (845° C.), loses its strength in a regular manner with the increasing grain growth of the submicroscopic ferrite particles at higher drawing temperatures. There is no rise to a peak of strength unless this has taken place at temperatures below 212° F. (100° C.). This failure of the strength to rise with increasing draws at this hardening temperature might be partially due to the lack of carbon in solution in the martensite. It has been shown by the microstructure that considerable masses of the original carbides remained undissolved at 1550° F. (845° C.). The specimens hardened at higher temperatures which have dissolved nearly all of the cementite present and increased the initial grain size show low initial strength, which rises to a peak with increasing drawing temperatures. Such a rise might be due partially to the conversion of retained austenite to martensite but in a greater degree to the growth of the cementite particle size to the critical size which produces the maximum keying effect. When this critical size is passed, the further increase in growth of the cementite particles, together with the concurrent increase in growth of the ferrite grains, produces a steady weakening of the structure.

In general it appears that the Rockwell hardness of the martensite and its decomposition products may be satisfactorily explained on the basis of the size and the growth of the submicroscopic ferrite particles alone. There is here no evidence that the critical dispersion of the cementite has a controlling effect upon the Rockwell hardness. In the control of the strength and the plasticity, the size of the precipitated cementite particles would appear to be the dominant influence due to the degree of their slip interference. The peak of plasticity comes at an early stage in the growth of the cementite particles where they exert a minimum of slip interference, while the peak of strength occurs when the particles have grown larger to the point where they exert a maximum of slip interference. In the case where the peaks of strength and plasticity roughly coincided, this result is believed to have been due to different portions of the martensite starting to break down at different times. It then became possible for half of the crystalline grains of decomposing martensite to arrive at the point of maximum strength while the other half were still at the point of maximum plasticity. The resultant effect was then a broadening of the peaks so that they might roughly coincide at some point. It is obvious that such a condition could produce neither the maximum strength nor the maximum plasticity of which the steel is capable. It must be recognized that the properties herein measured are only the summation of all of the properties of all the individual crystalline grains in the specimen. For example, in the specimens hardened at 1850° F. (1010° C.) and drawn from 500° F. (260° C.) to 600° F. (315° C.) the heights of the peaks of both plasticity and strength which roughly coincide are much below the maximum produced by other heat treatments.

Study of this theory as to the development of hardness, strength and plasticity indicates that the hardness will not be proportional to the strength except as a coincidence. It is perfectly possible to have maximum hardness and maximum strength at the same time. On the other hand, maximum plasticity will not occur with maximum strength nor with maximum hardness, in this particular steel.

## SUMMARY AND CONCLUSIONS

The type of steel under investigation has been long used in the fields of cutting tools, gages, bearings, etc. In such work its performance has been characterized by great ability to resist wear and a high degree of toughness. In this investigation it has been found very responsive to variations in heat treatment. It is possible by suitable changes in the hardening and drawing temperatures to secure many interesting combinations of properties. These different sets of properties in turn may be expected to fit the steel for many fields of usefulness. The flexibility of application has without doubt greatly assisted in promoting the commercial popularity of this steel. The numerical data here recorded will be of assistance in the future utilization of this steel and also serve to form a basis for comparing the properties of new and untried steels with a steel of well proven usefulness.

The data herein presented are believed to have a fundamental bearing upon the general theories of the hardening of steel. Some such theories have in fact assumed that hardened steel has no plasticity. The existence of a definitely measurable amount of plasticity is here demonstrated even in the case of steel in a condition which can be characterized as very brittle. The correlation of the results for hardness, strength, plasticity and microstructure has furnished new light upon the relations of these properties. Within the limits of this experiment the hardness of martensite and its decomposition products would appear to be adequately explained by the growth of the submicroscopic particles of ferrite. The strength is affected to some extent by the particle size of the ferrite but more largely by the keying effect of the precipitated submicroscopic particles of the iron carbide, cementite. The plasticity appears to be chiefly affected by the size of the submicroscopic cementite particles although the distribution of massive cementite also exerts a definite influence.

The method of testing herein described is believed to have demonstrated its usefulness for the determination of the properties of strength, plasticity and toughness in hardened tool steels. In fact, it appears possible that the use of the method may be extended to include other materials which are so brittle as to make determinations of plasticity or toughness difficult. While it is not assumed that the commercial value of a steel can be determined by these tests alone, they are believed to throw additional light upon the subject. At least it is now possible to obtain a general knowledge of the response of the mechanical properties of hardened tool steel to variations in heat treatment, before attempting to utilize it for a definite purpose. From this knowledge it is possible to plan heat treatments to definite ends so that the available combinations of properties may be used to the best advantage.

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(14) J. V. Emmons, "Structure and Heat Treatment of Tool Steel," *The Iron Trade Review*, Vol. 50, p. 450 (1912).

# DISCUSSION

MR. R. H. HARRINGTON[1] (*presented in written form*).—Mr. Emmons is to be congratulated for the fine work reported in this paper. The term "toughness" has long been used to indicate, in varying degrees, a combination of certain desirable properties. Some investigators have attempted to measure toughness by tests of a single property such as the reduction of area in a tension test or the various impact tests. Mr. Emmons' use of the toughness coefficient as combining strength with deformation may become a step in the evolution of some standard numerical means of comparing different degrees of toughness. It would be interesting to have further data in order to compare impact tests and body values with the various toughness coefficients for the different heat treatments.

MR. E. T. ICKES[2] (*presented in written form*).—Mr. Emmons has contributed an interesting and valuable paper. The definite manner in which the fact that hardness is not proportional to strength, except as a coincidence is brought out, is worthy of note. It is assumed that by strength, combined tensile strength and ductility, or in other words toughness, is meant.

We are glad to add as corroboration of his findings, that we have also noted in straight 1-per-cent carbon steels containing no alloys, a similarity in this respect. Further, that it is equally true, certain definite structural conditions are essential in securing maximum toughness and also maximum combined hardness and toughness.

In Mr. Emmons' experiments, it is noted that a time factor of 60 seconds was used exclusively for soaking at the hardening temperature. When working with steels containing 1 per cent of chromium or more, time, as well as temperature, is a very important factor in securing complete solution of the carbides, and their uniform diffusion in the matrix. It would be of interest to learn if Mr. Emmons has any data as to physical properties of this steel when held for longer periods at such hardening temperatures as 1500, 1550 and 1600° F. (815, 845, 870° C.).

In reference to the point that "for the higher hardening temperatures, the strength and hardness are inversely proportional until after the peak has been passed"—is this not true of all steels and to be expected in high-temperature hardening?

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[1] Research Metallurgist, General Electric Co., Schenectady, N. Y.

[2] Metallurgical Engineer, Columbia Steel and Shafting Co., Pittsburgh, Pa.

In the comments on Fig. 18, page 71, we question that the low plastic deformation is due to the precipitated particles of cementite, as our own work and that of others along these lines would indicate that the troostitic structure alone would produce greatest resistance to slip. Any effect of precipitated cementite particles would tend to increase slip rather than slip interference.

In the last paragraph on page 71, it can be readily understood that the coincidence of agreement of the peaks for strength and plastic deformation is due to the heterogeneity of the structure. Are we to understand from the succeeding statement that martensite has been definitely classed as a form of solid solution containing varying amounts of carbon? Could not the varying amount of precipitated cementite be due to insufficient diffusion time? Inadequate time allowance for diffusion would also account for the massive cementite particles of large size noted. The effect obtained by holding the specimens for longer periods at hardening temperatures might throw some additional light on this phase.

Mr. Emmons is to be congratulated not only for a very interesting paper, but also the avenues it opens for further study.

MR. H. H. ASHDOWN[1] (*presented in written form*).—This paper has added much useful information to our knowledge of tool steels, particularly in respect to the method of treatment for its different applications when comparing hardness to toughness or where a combination of these two properties is required.

It is a little unfortunate that the cross-sections of the specimens are so small, as one may expect different microstructural results and consequently different physical properties when treating larger sections and particularly those of irregular shapes.

One appreciates that it is less difficult to offer suggestions or criticism than to lay out and follow a program of research, but from a steel treater's point of view, the value of the paper would have been enhanced had it been shown, after complete solid solution had been obtained, in place of quenching immediately, to allow the temperature to fall somewhat and quench only at that temperature which is sufficient to maintain solid solution. This is a point of much importance, as in large or irregular sections the quenching stresses are considerably minimized.

The "blue brittleness" to which attention has been drawn on page 60 is of great interest and deserving of still closer investigation. It would appear to correspond closely with the very noticeable fall in Izod impact values very pronounced in certain chromium steels within a restricted low tempering range. It also indicates a zone where it is fatal to attempt to straighten tools or forgings.

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[1] Section Engineer, Material and Process Engineering Department, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

It is hoped the valuable information obtained by the author in this research will inspire him to continue his efforts for the development of the alloy tool steel industry.

MR. HOWARD SCOTT.[1]—Mr. Emmons has certainly given us some new things to think about, particularly as regards the effect of quenching temperature. It is surprising that he can use such very high quenching temperatures and still get a high degree of ductility in the material, and that at the high quenching temperature he uses, the structure does not coarsen up, apparently until a temperature of 1850° F. (1010° C.) is reached. I should like to ask if the absence of grain growth up to very high temperatures is a uniform characteristic of this type of steel? That is to say, is it representative of all steels of this type, or is it just particularly good steel with which he is working? It is an unfortunate characteristic of most of the carbon tool steels that they coarsen up and get brittle at very much lower temperatures. If Mr. Emmons has any suggestions as to what causes and controls that characteristic of steels they would be most welcome.

The high ductility that Mr. Emmons gets with even so high a quenching temperature as 1750° F. (955° C.) apparently reaches a peak at a tempering temperature which corresponds to the breakdown of austenite. The development of high toughness coincident with the disappearance of austenite appears to be quite significant. Possibly Mr. Emmons has some explanation of it?

Mr. Emmons made the point that his coefficient of toughness, the product of ductility times ultimate strength, is not a scientific property of the material. I should like to discuss that point with him, because I think we can consider the method scientific which aims at getting the optimum combination of properties even though the individual properties may be weighted arbitrarily. Certainly one desires both, high strength and high ductility, so I think he is quite justified in multiplying the values together and in picking out optimum compositions or treatment on that basis.

MR. H. J. FRENCH.[2]—The studies Mr. Emmons has made relative to the toughness of tool steels are really well worth while and the results have a definite meaning, but I should like to call attention to the fact that comparisons in so-called static tests do not necessarily parallel those of impact tests. Suddenly applied loads are frequently encountered in the application of steels in cutting tools, and therefore I should like to suggest to Mr. Emmons that it would be useful in future work to supplement the static test by some dynamic tests, especially notched-bar impact tests.

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[1] Section Engineer, Research Laboratories, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[2] Metallurgist, Development and Research Department, International Nickel Co., Bayonne, N. J.

There are some practical difficulties in attempting to apply the customary Izod or Charpy impact tests to tool steels which have been hardened and either not drawn at all, or drawn at only slightly elevated temperatures, but such tests can yield useful comparisons if made on un-notched specimens or on round specimens with notches with $\frac{1}{8}$-in. radius, $\frac{1}{8}$ in. deep; but along with this modified notch a hammer should be used which is only a fraction of the total weight of the standard hammer on the Izod testing machine. Tests made under such conditions enable distinctions to be made between different steels and different treatments, and have been useful particularly when combined with other tests indicative of strength or hardness in appraising what we sometimes call "toughness."

I hope Mr. Emmons will find it possible to continue his researches so as to include other steels than those which he has already examined.

MR. M. A. GROSSMANN[1] (*by letter*).—The author is certainly to be complimented on the ingeniousness of his torsion test. As he points out it permits taking a measurement of yielding over a long specimen. This is an extraordinary advantage over the impact specimens we formerly employed to get an index of the toughness.

The theory worked out seems to me to be well supported by the evidence with possibly one exception on which I should like to hear an opinion. This particular point is the evidence that the austenite in such steel is for the most part decomposed only at temperatures of 450 to 650° F.

Shrinkage measurements on hardened steel of this kind show that when it is tempered, it contracts at reheating temperatures up to about 400° F. and that reheating beyond this range causes slight expansion, followed by very pronounced contraction when temperatures of 700° F. and above are reached. It is in this range of 450 to 650° F. that impact tests on such steel show brittleness. It had seemed to me that the alpha iron from decomposition of austenite in this range might account for the brittleness observed. Furthermore, in the data given in the paper there is irregularity in angular deformation in this same range, to which attention is called on page 60 of the paper and which is illustrated in Fig. 6. The low ductility in the range 500 to 700° F. would correspond with the brittleness found in impact tests. In other words, the expansion on tempering around

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[1] Vice President and Director, Republic Research Corp., Massillon, Ohio.

450 to 650° F., the impact brittleness in this range, and the low ductility in the torsion tests in this same range, all point to austenite being decomposed at 450 to 650° F. instead of at 350° F. as suggested.

I have an impression that this is also supported by X-ray evidence by Bain and also by Heindlhofer.

As stated before this is the only point I would raise in what seems to me to be a thoroughly substantiated theory.

MR. A. F. WESTGREN[1] (*by letter*).—The paper by Mr. Emmons has been read with much interest. The comparison between hardness, strength and toughness is very instructive, indeed, but I think the theoretical explanation of the change of these properties with quenching and drawing temperature is a very difficult problem. It is suggested (page 68) that the retained austenite is transformed into martensite after a draw of 260° C. (500° F.). This agrees perfectly with the results of B. D. Enlund,[2] who also has proved that there is some austenite retained in all quenched carbon steels.

My opinion on the reactions taking place on drawing of a quenched steel is very much the same as that put forward in a paper[3] which has been published quite recently by my collaborator Mr. Ohman. He has communicated his results also at the last meeting of the British Iron and Steel Institute.

MR. F. B. LOUNSBERRY[4] (*by letter*).—I would offer the opinion, after an examination of the specimens, that the large undissolved cubical constituents such as appear in Figs. 12, 13, 14, 17 and 21 of Mr. Emmons paper, are zirconium nitride crystals.

MR. J. V. EMMONS[5] (*author's closure by letter*).—The author wishes to express his thanks to those who have so kindly contributed their thoughts as to the results of the paper and their suggestions as to profitable avenues for future work.

In reply to the particular queries: When the term "strength" is used in the paper it is with the meaning of ultimate strength and is not to be confused with "toughness" which is regarded as a combination of the properties of strength and plasticity.

In regard to the effect of troostite and precipitated particles of cementite upon plasticity, the author's interpretation of the results is that for the steel and conditions of these experiments, troostite promotes plasticity and there is a certain particle size of cementite which produces a maximum of slip interference.

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[1] University of Stockholm, Stockholm, Sweden.

[2] *Journal*, Iron and Steel Inst. (British), Vol. I (1925).

[3] *Jernkontorets Annaler*, p. 325 (1931).

[4] Vice-President and Works Manager, Atlas Steel Corp., Dunkirk, N. Y.

[5] Metallurgist, The Cleveland Twist Drill Co., Cleveland, Ohio.

In the decomposition of austenite, it is difficult to fix an exact point at which the solid solution begins to precipitate carbide particles. It appears probable that the alpha iron, when first formed, is highly supersaturated with carbon. The first precipitation of carbon is believed to be in particles of colloidal size which coalesce into larger particles whenever there is sufficient freedom of movement. It is believed that the original solid solution is not homogeneous in its carbon content and contains carbon-rich and carbon-poor areas which decompose at different rates. The result is an intimate mixture of the different decomposition products in a fine state of division. The physical properties as determined by the methods described in the paper are in reality an average of the properties of all of the constituents present.

The peaks of toughness are not believed to be due to the disappearance of the austenite, but rather to the release of strains and the precipitation of the carbide on a particle size too small to key the crystallographic planes against slipping.

Chromium steels of the type under consideration generally show great resistance to grain growth at high temperatures. The steel here used is believed to be representative of its type.

The evidence that austenite is decomposing to martensite at temperatures from 350 to 500° F. is principally based on the results for hardness. The exact temperature range in which austenite decomposes is affected by such variables as chemical composition and hardening temperature, so that comparison with the work of other authors is difficult.

It is hoped that at some future time an exact correlation may be made of the results of X-ray analysis and the physical properties studied in this paper. Such information if obtained from the same series of specimens might greatly improve our knowledge of the decomposition of austenite. If still other physical properties such as volume change and the magnetic properties could be studied simultaneously, the possible deductions would be still more conclusive.

It is also hoped that a future study may include the effect of other variables such as chemical composition, soaking times at the hardening and drawing temperatures, quenching speed and the size of the specimen.

# THERMOMAGNETIC ANALYSIS AND THE $A_0$ TRANSFORMATION IN 0.75-PER-CENT CARBON STEEL[1]

By R. L. Sanford[2] and G. A. Ellinger[3]

## Synopsis

The paper gives a brief outline of the methods of thermomagnetic analysis, a short discussion of the commonly accepted idea of the nature of the $A_0$ transformation and a description of an investigation on the influence of particle size and distribution of iron carbide on the magnetic $A_0$ transformation in the case of a 0.75-per-cent carbon steel. The fact that the intensity of the magnetic transformation increases with decreasing particle size would seem to show that the presence of the carbide is surely indicated by thermomagnetic analysis however finely dispersed it may be.

It is pointed out that, properly carried out and interpreted, thermomagnetic analysis is of great promise as a method for studying the structure of steel and the transformations associated with its heat treatment.

## Introduction

In his study of the constitution and structure of steel and the effect upon them of thermal and mechanical treatment, the physical metallurgist does not depend upon the metallurgical microscope alone. He also utilizes various physical measurements for the determination of properties which may serve to indicate the conditions of the material under consideration. Furthermore, there are certain changes or transformations associated with the hardening and tempering of steel which can be observed by suitable methods. Thus, thermal analysis, X-ray analysis and dilatometric measurements have been employed as well as mechanical tests of strength, hardness, ductility, and the like. No single method is capable of telling the whole story, but the combined evidence obtained by many methods must be correlated and harmonized in order to arrive at definite and valid conclusions. Among the methods available for the study of structural transformations which take place during the heat treatment of steel is thermomagnetic analysis by means of which changes in magnetic properties are observed as the material under investigation is heated or cooled. This method has been developed and used quite extensively by Honda[1] and his associates in Japan but except for isolated researches here and there has been rather neglected elsewhere.

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[1] Publication approved by the Director of the Bureau of Standards of the U. S. Department of Commerce.

[2] Chief, Magnetic Section, U. S. Bureau of Standards, Washington, D. C.

[3] Assistant Metallurgist, U. S. Bureau of Standards, Washington, D. C.

Although it is possible that, in some instances, the conclusions drawn from the results of thermomagnetic analysis may have been more far reaching than the data would warrant, nevertheless, this method, properly applied and correctly interpreted, constitutes a very powerful tool for the study of phenomena some of which are beyond the scope of the other methods. Obviously, it is necessary not only to develop a satisfactory technique but also to determine fundamental data to serve as criteria for the correct interpretation of results. The data here presented cover one phase of a comprehensive study undertaken primarily for the purpose of obtaining such fundamental data.

## THERMOMAGNETIC ANALYSIS

Thermomagnetic analysis may be carried out either by making more or less complete magnetic tests at a series of fixed temperatures held constant during the magnetic tests, or continuous observations of magnetization may be made under a definite condition such as a steady applied magnetizing force while the material is being heated or cooled at a controlled rate. Each of these procedures has its own peculiar advantages and limitations, and the method to be followed in a given case depends largely upon the purpose for which the data are required and the nature of the material under investigation. Much more complete data can be obtained by the first named method, but it is often undesirable to hold the specimen at a fixed temperature long enough to make the complete magnetic tests. This is particularly true when the specified temperature lies within the range in which a structural transformation takes place. It is also possible to miss entirely changes occurring at temperatures intermediate between those at which observations are taken. It is generally preferable, therefore, especially when structural transformations are under investigation, to make continuous observations of magnetic changes which take place during heating or cooling.

The most satisfactory method for the continuous observation of magnetization during the process of heating or cooling is by means of the magnetometer. Some experimenters prefer to use alternating-current methods because of the sensitivity of the magnetometer to extraneous influences. It is possible, however, by proper design and installation of the apparatus to reduce the effect of outside influences to a negligible magnitude. By the use of the magnetometer it is possible to observe variations of quantities such as remanent induction and coercive force which would be difficult or impossible to measure with alternating currents and there are no complications due to skin effect and phase differences.

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[1] The work of Honda and his associates covers a wide range in this field and for the most part is reported in numerous papers in *Science Reports*, Tohoku Imperial University, Sendai, Japan.

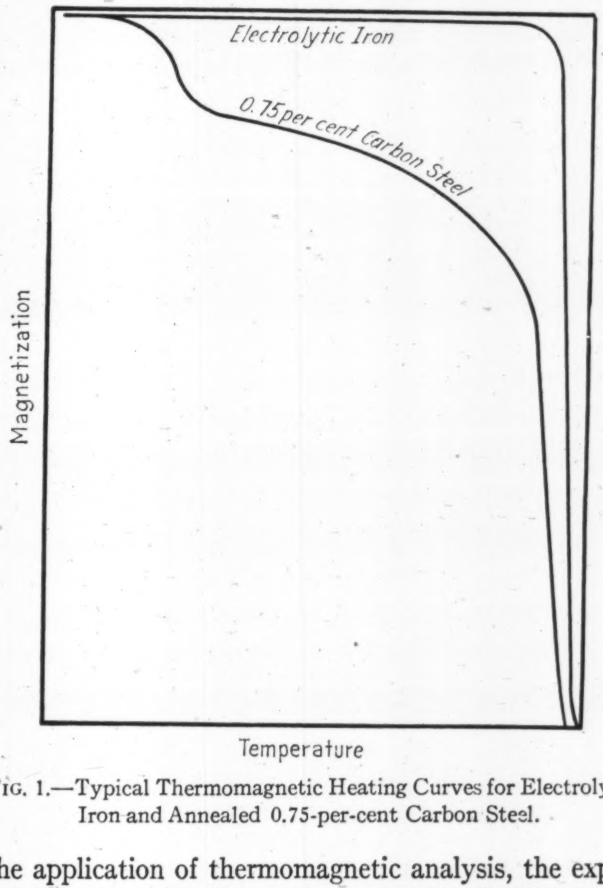

FIG. 1.—Typical Thermomagnetic Heating Curves for Electrolytic Iron and Annealed 0.75-per-cent Carbon Steel.

In the application of thermomagnetic analysis, the experimental conditions must be taken into consideration in the interpretation of results. For example, the normal reaction of pure iron to heating under the influence of a steady applied magnetizing force is quite different at low values of magnetizing force from that at high values. In general, the effect of strain and other sources of variability of magnetic properties is greater in the second stage of magnetization represented by the steep part of the magnetization curve than in the third stage or upper part of the curve. It is preferable, therefore, to work in the third stage of magnetization when structural transformations are under investigation. The proportions of the specimen have an important effect on the measurements because of the self-demagnetizing effect. This effect depends upon the ratio of length to diameter which should preferably be not less than 15. The length of specimen that can be used is limited by the characteristics of the magnetizing coil and furnace. It is undesirable to have the specimen longer than the region in which the magnetizing force and temperature are practically uniform.

## THE $A_0$ TRANSFORMATION

The magnetization of pure iron under the influence of a steady magnetizing force of suitable value does not change appreciably upon heating until a relatively high temperature is reached, usually of the order of 500 to 600° C. (930 to 1110° F.). An annealed high-carbon steel, on the other hand, shows a drop beginning at approximately 150° C. (300° F.) and ending at about 215° C. (420° F.). Curves for pure iron and high-carbon steel are shown in Fig. 1. The drop, in the case of the steel, appears to be due to a change taking place in some constituent of the material and is attributed to a magnetic transformation of the iron carbide, $Fe_3C$. Honda[1] has rather conclusively shown that this is the case and has determined that the end of the transformation on heating or its start on cooling occurs at 215° C. The transformation is generally called the $A_0$ transformation.

A hardened carbon steel shows no trace of the $A_0$ transformation on heating. This may be taken to mean either that no carbide is present or that, because of the fineness of the carbide particles, they do not contribute appreciably to the magnetization. In view of the present uncertainty as to the condition of the carbon in a quenched carbon steel, it is important to know which of these two interpretations is correct. The present investigation was undertaken, therefore, with the primary object of determining the influence of carbide particle size on the $A_0$ transformation with the end in view of deciding whether or not the absence of the $A_0$ transformation is sufficient evidence that iron carbide, $Fe_3C$, is not present in untempered martensitic steel.

## APPARATUS AND MATERIAL

The apparatus used was the same, except for certain minor modifications, as that described some time ago by one of the present authors.[2] The specimens were magnetized in a magnetizing coil 36 cm. long within which is mounted a water-jacketed platinum resistance furnace wound non-inductively so that the heating current has no appreciable magnetic effect on the specimens. This coil is set with its axis at right angles to the earth's magnetic field. The winding is such that the magnetic field at its center is approximately 79 times the current in amperes.

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[1] *Science Reports*, Tohoku Imperial University, Sendai, Japan.

[2] R. L. Sanford, "Apparatus for Thermomagnetic Analysis," *Bureau of Standards Journal of Research*, Vol. 2, Research Paper No. 50, p. 659 (1929).

The degree of magnetization of the specimen is indicated by an astatic magnetometer whose deflections are read by means of a lamp and scale. The magnetometer is mounted west of the magnetizing coil and in line with its axis. On the opposite side of the magnetometer is a compensating coil connected in series with the magnetizing solenoid and so placed that its magnetic effect on the magnetometer just neutralizes that of the magnetizing coil. The magnetometer deflections are thus proportional to the magnetization of the specimen. Temperatures are read by means of a platinum, platinum-rhodium thermocouple and Leeds & Northrup, Type K, potentiometer. The heating current is controlled by a specially designed autotransformer.

A specimen of 6 mm. diameter was chosen so that, if desired, other tests could be made with apparatus requiring this diameter. It is desirable that the ratio of length to diameter be at least 15 in order to avoid an excessive self-demagnetizing effect. A length of 10 cm. makes the ratio 16.7 and is not too great to obtain uniformity of temperature and magnetizing field in the apparatus. The specimens were therefore made 10 cm. long and 6 mm. in diameter with a small hole drilled longitudinally in one end to permit insertion of the thermocouple.

The rods from which the samples were made were originally $\frac{3}{8}$-in. (0.95 cm.) in diameter and were either cold drawn to size before heat treatment or were machined to size after heat treatment. The latter method was probably better as there could then be no question of surface decarburization. Also, it was found that by this procedure the disturbing effects of internal strain were practically eliminated.

The material was a 0.75-per-cent carbon steel having 0.52 per cent manganese, 0.018 per cent phosphorus, 0.033 per cent sulfur and 0.12 per cent silicon and was subjected to various heat treatments designed to give the desired differences in particle size and distribution of carbide.

## OBSERVATIONS AND RESULTS

Observations were made on three sets of specimens prepared in different ways. Temperature-magnetization curves were made only on heating and the temperature was not carried beyond 300° C. (570° F.) as the $A_0$ transformation only was under investigation. The heating rate was slow, generally not over 2° C. per minute. No difficulty due to extraneous magnetic influences was experienced as the magnetometer was astatic to such a degree that the effect of outside disturbances was entirely negligible.

The applied magnetizing force was in all cases 277 gilberts per centimeter. The effective value depends upon the magnetization of the specimen but it is estimated that in the extreme case the effective magnetizing force was not less than 150 gilberts per centimeter. For the material under investigation, this value is sufficient to bring the specimens beyond the magnetically unstable second stage of magnetization into the more stable third stage. As previously pointed out, observations taken in the second stage or steep part of the magnetization curve are difficult to interpret because, in this region, an increase in permeability always accompanies an increase in temperature and there is no way of differentiating between this normal rise at the beginning of the heating curve and an increase that might be brought about by the relief of internal strain. This is an important point which should not be overlooked in the application of thermomagnetic analysis.

For the first set of observations, specimens of $\frac{3}{8}$-in. rod were quenched from 1016° C. (1860° F.) into hot lead at 593° C. (1100° F.) and then air cooled to room temperature. This treatment is commonly called "lead patenting" in the wire drawing industry. After patenting, the rods were cold drawn to a diameter of 6 mm. (0.24 in.) and cut into specimens 10 cm. (3.9 in.) long. The structure was examined under the microscope and found to be predominantly sorbitic with a small amount of pearlite present. In order to spheroidize the carbide and produce carbide particles of various sizes, groups of specimens were held at a temperature of 680° C. (1255° F.) for different lengths of time varying from 15 minutes to 64 hours.

Figure 2 shows curves for five of the specimens. In all of these curves there is an initial rise in magnetization due no doubt to relief of the strains set up by the cold-drawing process. In the unspheroidized specimen, this effect is so great as to mask entirely the $A_0$ transformation. The spheroidizing treatment reduces the effect progressively as the time at temperature is increased until for the 4-hour specimen (not shown) it is entirely absent.

It will also be noted that the intensity of the $A_0$ transformation seems to decrease as the time of holding at temperature is increased thus increasing the size of the carbide particles. The temperature at which the transformation appears to be complete is lower the longer the time of holding at the spheroidizing temperature. The curves cannot be interpreted with certainty, however, on account of the varying effect of strain superposed on the effect of the change in structure.

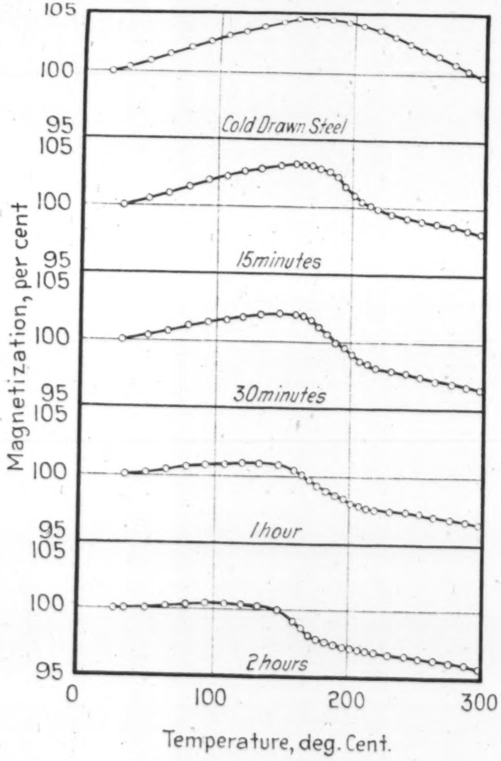


FIG. 2.—Thermomagnetic Heating Curves for Cold-Drawn 0.75-per-cent Carbon Steel Wire, Held at 650° C. for the Lengths of Time Indicated.

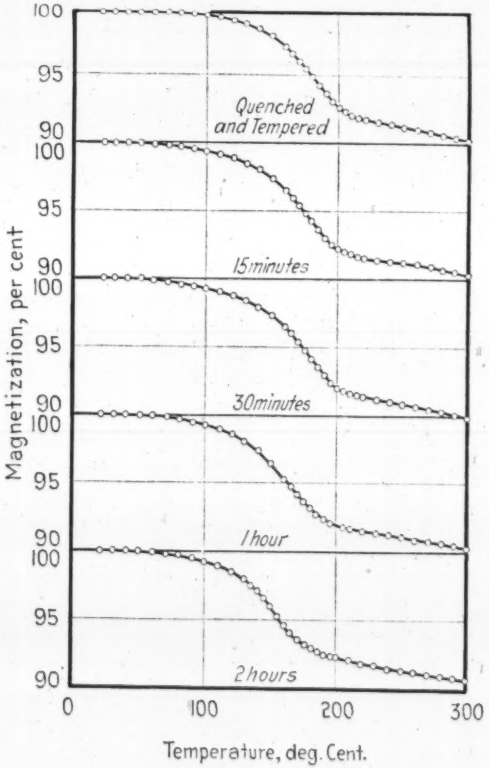


FIG. 3.—Thermomagnetic Heating Curves for 0.75-per-cent Carbon Steel, After Holding at 680° C. for the Lengths of Time Indicated.

The specimens were originally quenched from 950° C. in cold water and tempered at 575° C.

A second set of specimens was prepared in a somewhat different manner with the object of minimizing the influence of strain and also of avoiding any possible effect due to surface decarburization. The rods were patented as before, and then machined to size before spheroidizing. The spheroidizing temperature for this set was 720° C. (1330° F.). The thermomagnetic curves for this set showed an $A_0$ transformation of much greater magnitude and more characteristic of a pearlitic structure. Microscopic examination of the 2-hour specimen at a magnification of 2000 diameters revealed a structure of very fine pearlite. Further work with this set was therefore abandoned.

Instead of patenting the third set of specimens, they were quenched from 950° C. (1515° F.) into oil at 20° C. (68° F.) and then tempered at 575° C. (1065° F.). They were then machined to size and spheroidized at 680° C. (1255° F.). Microscopic examination after tempering showed a sorbitic structure with no pearlite present.

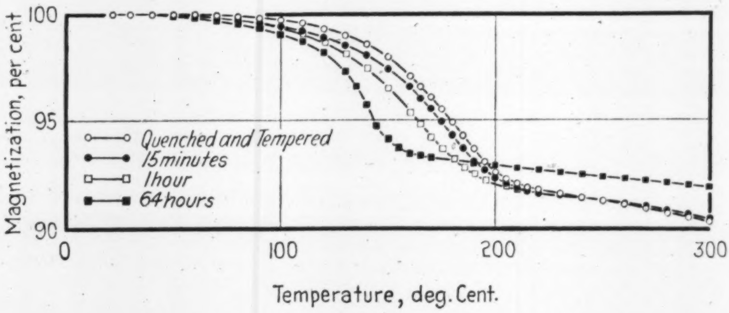


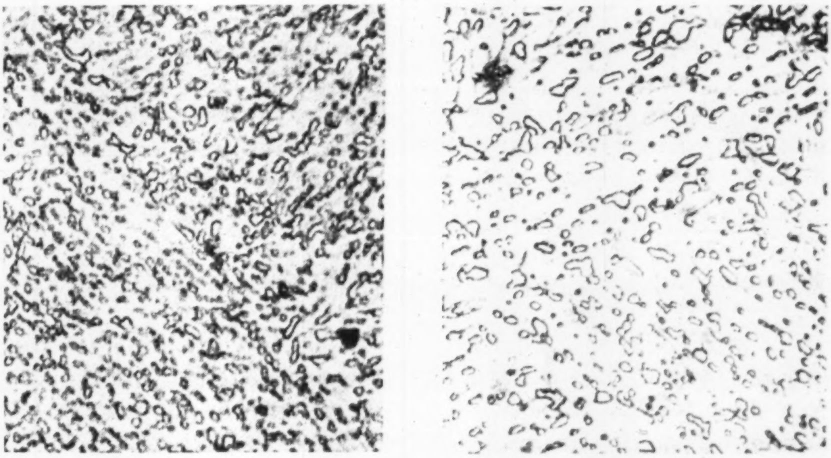
FIG. 4.—Same as Fig. 3, with the Original Quenched and Tempered Specimen Together with Those Held at 680° C. for 15 minutes, 1 hour and 64 hours, Superposed to Show the Apparent Decrease in Intensity and Temperature of the $A_0$ Point as the Carbide Particles Become Larger.

The thermomagnetic curves given in Fig. 3 do not show the initial increase due to strain. The transformation occurs at lower temperatures as the carbide particle size increases and its intensity is less the larger the particles. This is shown more clearly in Fig. 4 in which the curves for the original sorbitic, the 15-minute, 1-hour and 64-hour specimens are superposed.

All of the specimens in the last set were examined under the microscope in order to be sure that the desired growth in carbide particle size had actually been obtained. Two of the micrographs are reproduced in Fig. 5. They show the structure at a magnification of 2000 diameters for the 2-hour and 32-hour specimens. It is plainly evident that the desired growth was obtained.

## DISCUSSION OF RESULTS

In discussing the results of the present investigation, it should be pointed out at the outset that the work was carried out on a single composition of steel and therefore too sweeping and general conclusions should not be drawn from the results. There are many questions remaining which must be answered before the results of thermomagnetic analysis can be interpreted with certainty for all classes of steel. The results do furnish, however, what appears to be fairly strong evidence on the question primarily under consideration, that is, whether or not the absence of the magnetic $A_0$ transformation should be taken as evidence that no iron carbide is present in the structure.



(*a*) Specimen after holding at 680° C. for 15 minutes.

(*b*) Specimen after holding at 680° C. for 32 hours.

FIG. 5.—Photomicrographs of Two Quenched-and-Tempered Specimens. Etched in 2-per-cent Nital (× 2000).

That the $A_0$ transformation is due to the carbide is generally accepted. Whenever the transformation has been observed, the conditions have been such that the carbide was surely present. On the other hand, no $A_0$ transformation is observed in the case of pure iron in which there is no carbide. These facts in themselves, however, cannot be taken as proof that whenever the carbide is present the $A_0$ transformation will be found. It is entirely conceivable that the carbide might exist under such conditions as, for example, very fine subdivision that it would have a negligible effect on the magnetic properties of the material as a whole or that, due to influences such as strain, the temperature of the transformation might be altered. This question cannot be answered by experiments on quenched steel because it is not known from examination by other methods whether such material contains the carbide or not. It was for this reason that the initial structure chosen was sorbite in which the carbide is known to exist in very fine particles. By starting with such a structure and then modifying the carbide particle size by suitable heat treatment, it should be possible to discover the trend of the change in the intensity of the $A_0$ transformation with change in particle size.

In the original sorbitic structure no trace of pearlite was found, but, as shown in the upper curve of Fig. 3, there is a well-defined $A_0$. It appears therefore that carbide does not have to be associated with a pearlitic structure in order to exhibit the transformation. As the carbide particles increase in size, the intensity of the $A_0$ transformation does not increase but definitely decreases. This is shown more clearly in Fig. 4. Stated conversely, the intensity of the $A_0$ transformation increases as the carbide particle size decreases in a non-lamellar structure. Although it may not be safe to carry the extrapolation to the limit, the indication is that mere fineness of carbide particle size is not sufficient to suppress the $A_0$ transformation. In the absence of proof to the contrary, therefore, it is believed that the absence of an $A_0$ transformation in hardened steel is indicative that no carbide is formed during the quenching process.

The decrease in the temperature at which the $A_0$ transformation appears to be complete as the carbide particle size is increased is not easy to explain. Honda[1] has rather conclusively shown that the transformation of cementite itself is completed at 215° C. (420° F.) and this is the temperature usually found when the structure is mainly pearlitic. The magnetization continues to decrease with increasing temperature after the $A_0$ transformation appears to be complete, even though under similar conditions pure iron does not begin to show a decrease till much higher temperatures are reached. The carbide apparently has some influence on the thermomagnetic properties of the material even above the temperature at which it becomes practically non-magnetic. Much further work on this point needs to be done before a definite decision can be reached.

## CONCLUSIONS

The chief object of the present investigation was to determine the effect on the $A_0$ transformation of carbide particle size. This is an important point on account of its bearing on the reason for the absence of an $A_0$ in hardened carbon steel.

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[1] K. Honda and T. Murakami, "On the Thermomagnetic Properties of the Carbides Found in Steels," *Science Reports*, Tohoku Imperial University, Sendai, Japan, No. 6, p. 23 (1917).

It was found that:

1. The $A_0$ transformation takes place upon heating steel having a sorbitic structure and, therefore, pearlite is not necessary for the transformation.
2. The intensity of the $A_0$ transformation is greater the smaller the carbide particle size and, therefore, the absence of an $A_0$ transformation probably indicates the absence of the carbide.
3. The temperature at which the transformation appears to be complete is not constant for the non-pearlitic structure as it is for the pearlitic structure but depends upon the size of the carbide particles and is lower the larger the particles.

It is realized that much more work must be done, particularly with steels of various carbon contents before the question is conclusively settled, but the authors are of the opinion, in the absence of proof to the contrary, that the evidence of thermomagnetic analysis is strongly in favor of the view that untempered martensitic steel does not contain the carbide $Fe_3C$ in its structure.

# RELATION BETWEEN MAGNETIC PROPERTIES, IMPACT STRENGTH AND HARDNESS

By Haakon Styri[1]

## Synopsis

The results of an investigation of a subcommittee of the Society's Committee A-8 on Magnetic Analysis, formed to investigate the relationship between magnetic testing and impact strength and hardness are given.

The material selected for the investigation was of standard ball bearing steel analysis and was checked for freedom of defects before being made into test specimens. A number of heat treatments were chosen to cover the complete probable range of heat treatment for this type of steel. Magnetic tests were made on unnotched impact specimens before and after heat treatment. After completion of the magnetic tests, the specimens were notched with a grinding wheel and Charpy impact and Rockwell hardness tests made.

The results show that a number of magnetic methods of investigation gave direct correlation with hardness and impact strength and that the latter decreases with increase in hardness.

Variation in tempering temperatures in the range investigated have much greater influence on hardness and impact values as well as magnetic properties than changes in quenching temperatures.

The results are summarized in graphical form.

## Introduction

Previous reports of the Society's Committee A-8 on Magnetic Analysis[2] have described methods of magnetic analysis and have included correlation with hardness and cutting qualities. The investigations, however, covered steels of complicated analyses or samples of too irregular shape to be suitable to determine the magnetic properties. It was therefore considered desirable to determine fundamental magnetic properties in relation to the more recently developed magnetic testing methods and to certain other physical properties, using a simple shape of a commercial steel, heat treated over a wide range.

The subcommittee appointed to perform this work adopted a proposal made by Mr. F. P. Fahy, to make this investigation on specimens suitable for standard impact tests, made from a steel of the usual ball bearing analyses.

It was at first considered necessary, in order to facilitate magnetic tests, to use bars of multiple length of impact specimens with 10-mm. cross-section without any notches, and to cut the specimens to size after the magnetic tests. Preliminary tests indicated, however, that it would be very difficult to heat treat such long bars, especially in water, without considerable warpage, which would make it difficult to grind the specimens to size. Holes drilled in the specimens to limit the notch would also vary the cross-section and would cause bending of water-quenched specimen after notching. Tests made by Mr. R. L. Sanford showed that it was satisfactory to use a 60-mm. specimen without notching for the magnetic tests and it was found that a notch of suitable dimension could be ground in the specimen after heat treatment. All magnetic tests could therefore be made on specimens of full cross-section 10 mm. square and a length of 60 mm.

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[1] Director of Research, SKF Industries, Inc., Philadelphia, Pa.

[2] *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part I, p. 157 (1926.)

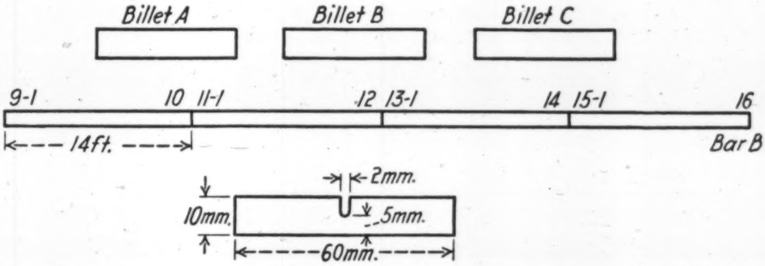

FIG. 1.—Showing Test Specimens and Method of Taking Specimens.

## MATERIAL AND PREPARATION

Through the courtesy of Mr. Howard, Simonds Steel Mills furnished the material from a basic electric heat of the following analyses: carbon 1.03 per cent, phosphorus 0.017 per cent, sulfur 0.010 per cent, silicon 0.27 per cent, manganese 0.33 per cent, chromium 1.50 per cent, and nickel 0.32 per cent.

Three 2-in. billets were rolled from a 600-lb. ingot and after examination for satisfactory structure and cleanliness, $\frac{1}{2}$-in. bars were rolled from the billets as shown in Fig. 1 and box annealed.

Specimens used in the tests were cut only from bar B at the SKF Research Laboratory and numbered on the end consecutively from the left of each of the 14-ft. lengths, 9–1, 11–1, etc. The specimens were machined to 11-mm. section and 60.5 mm. long, rough ground on all sides to 10.2 mm., checked magnetically for uniformity, and hardened by Mr. Walp according to the groups given in the table. The four specimens in each group were wired together, well separated, heated in a salt bath and held at the temperature of treatment ± 3° C. for 30 minutes and quenched in oil at a temperature between 33 and 43° C. or in water between 23 and 33° C. with vigorous stirring. Surface troostite due to quenching in water was expected to be eliminated in the final surface grinding. Small amounts were, however, found but did not noticeably effect the magnetic tests which indicated good uniformity. Some of the specimens, especially those quenched from the higher temperatures, developed surface tempering and slight cracking in the final grinding to 10 mm., in spite of great care, probably due to the necessity of grinding them in the untempered condition. Such specimens were replaced by others which were heat treated separately and can be identified in the table by their irregular numbers.

After grinding to accurate length, the specimens were tested magnetically and tempered in groups in a Homo furnace for 30 mintues at the tempering temperature ± 2° C.

## TESTS

Magnetic tests were performed on the specimens by all interested members of the subcommittee.

Mr. Sanford reports as follows:

*Preliminary Magnetic Tests.*—The first test of the specimens before heat treatment were made with a simple alternating-current method using two similar magnetizing solenoids connected in series. Within the two solenoids were similar test coils of 1000 turns each. These were connected in series-opposition to an alternating-current galvanometer whose field was excited from the same source as the magnetizing coils. A specimen which served as a standard was placed in one solenoid and the others compared with it in terms of the galvanometer deflection. A 60-cycle source was used and tests were made at both low and high values of current. The specimens were found to be exceedingly uniform. The alternating-current test was amply sensitive, as tests with the Fahy Simplex permeameter using the high magnetization adapter showed no appreciable difference between the two specimens showing the greatest difference by the alternating-current test.

The specimens were then returned to the SKF Research Laboratory where they were hardened according to a schedule previously agreed upon. Magnetic tests were then made using the Simplex with adapter. $B_{\text{max.}}$, $B_r$, and $H_c$ were determined for each specimen using an $H_{\text{max.}}$ of 1000 gilberts per centimeter. The specimens in each group showed a satisfactory degree of uniformity and they were accordingly sent to SKF for tempering.

*Final Magnetic Tests.*—Before testing the tempered specimens in the Simplex adapter they were intercompared by the method of de Forest using the cathode ray oscillograph. This intercomparison showed that, in general, the specimens in each group having the same heat treatment were remarkably similar magnetically. Accordingly, quantitative tests were made on more than one specimen in a given group only when they appeared to be different by the oscillograph test. The results of the tests with the Simplex adapter are given in the tables. Figures are given for each specimen showing differences by the oscillograph.

*Discussion.*—The method used for testing on a quantitative basis is not so sensitive to slight differences in the specimens as may be some of the comparative methods. It remains to be seen, however, whether greater sensitivity is required. It is possible that too great sensitivity may lead to confusion and render interpretation somewhat difficult. It is of interest to note that oil quenching, at least for the lower temperatures, appears to give more uniform results than does water quenching. It would also appear safe to conclude, at least tentatively, that within the range of temperatures used there is no well-defined optimum quenching temperature. It will be necessary to await the

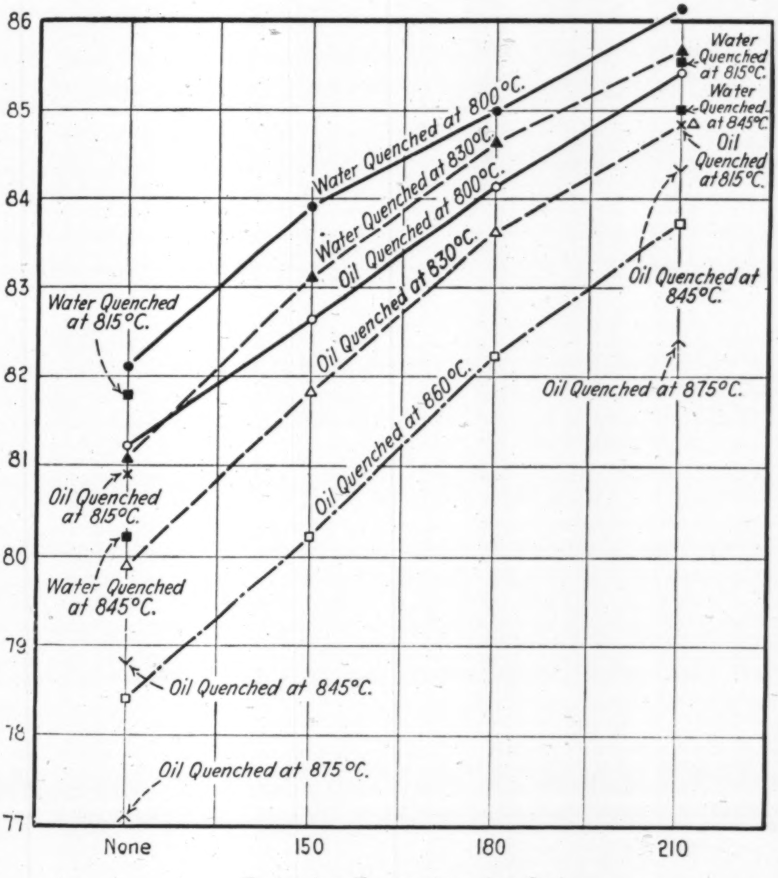

FIG. 2.—Showing Average Readings Obtained by A. V. de Forest on an Inductance Bridge.

TABLE I.—COMPARATIVE RESULTS, FOUR MAGNETIC METHODS, ROCKWELL HARDNESS, CHARPY IMPACT, ON BALL BEARING STEEL.

| Quenching Temperature, deg Cent. | Tempering Temperature, deg. Cent. | Specimen | Tests by Sanford | | | Tests by Kouwenhoven | | Tests by de Forest | Tests by SKF Laboratories | Rockwell Hardness | Impact, kg-cm. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | $M_1$ | $R_1$ | $C_F$ | $\theta_L$ | $\theta_M$ | | | | |
| | | | | | OIL QUENCHED | | | | | | |
| 800 | none | No. 9-1 | 17.27 | 10.42 | 42 | 347 | 323 | 82.2 | +15 | 62.6 | 33 |
| | | No. 11-1 | | | | 345 | 323 | 81.1 | +15 | 62.5 | 21 |
| | | No. 13-1 | | | | 346 | 323 | 81.4 | +15.5 | 62.6 | 29.5 |
| | | No. 15-1 | | | | 345 | 323 | 81.1 | +15 | 62.4 | 29.5 |
| | | Average | | | | 345 | 323 | 81.4 | +15.1 | 62.5 | 28.5 |
| | 150 | No. 9-2 | 17.35 | 10.66 | 38 | 352 | 327 | 82.5 | +18 | 61.4 | 22.2 |
| | | No. 11-2 | | | | 353 | 327 | 82.5 | +17 | 61.3 | 35.4 |
| | | No. 13-2 | | | | 351 | 326 | 82.7 | +18 | 61.5 | 35.4 |
| | | No. 15-2 | | | | 351 | 326 | .... | +17 | 61.5 | 36.2 |
| | | Average | | | | 351.7 | 326.5 | 82.6 | +17.5 | 61.4 | 32.2 |
| | 180 | No. 9-3 | 17.43 | 10.78 | 38 | 357 | 331 | | +21 | 60.8 | 32.8 |
| | | No. 11-3 | | | | 358 | 331 | | +20 | 60.5 | 35.4 |
| | | No. 13-3 | | | | 357 | 330 | | +20 | 60.5 | 32.2 |
| | | No. 15-3 | | | | 358 | 330 | | +20 | 60.6 | 39.5 |
| | | Average | | | | 357.5 | 330.5 | 84.1 | +20.2 | 60.6 | 35 |
| | 210 | No. 9-4 | 17.67 | 11.18 | 30 | 369 | 336 | 85.6 | +26 | 60.0 | 35.4 |
| | | No. 11-4 | | | | 367 | 335 | 85.6 | +25 | 59.8 | 40.2 |
| | | No. 13-4 | | | | (365.5)[a] | (334)[a] | (84.3)[a] | (+23)[a] | 59.9 | 34.1 |
| | | No. 15-4 | | | | 368 | 335 | 85.1 | +25 | 59.9 | 35.4 |
| | | Average | | | | 368 | 335.5 | 85.4 | +25.2 | 59.9 | 36 |
| 815 | none | No. 9-5 | 17.15 | 10.34 | 40 | 346 | 321 | 80.9 | +13 | 63.1 | 17.3 |
| | | No. 11-5 | | | | 345 | 321 | 81.2 | +13 | 63.0 | 20.3 |
| | | No. 13-5 | | | | 344 | 320 | 80.9 | +13 | 63.3 | 21 |
| | | No. 15-5 | | | | 344 | 321 | 80.5 | +13 | 63.4 | 19.7 |
| | | Average | | | | 344.7 | 320.7 | 80.9 | +13 | 63.2 | 19.7 |
| | 210 | No. 9-6 | 17.50 | 11.05 | 32 | 368 | 334 | 83.8 | +25 | 59.7 | 36.8 |
| | | No. 11-6 | | | | 368 | 335 | 86.1 | +24 | 59.9 | 53 |
| | | No. 13-6 | | | | 367 | 334 | .... | +24 | 60.0 | 37.5 |
| | | No. 15-6 | | | | 367 | 334 | 84.4 | +24 | 59.8 | 35.4 |
| | | Average | | | | 367.5 | 334.1 | 84.8 | +24.1 | 59.9 | 41 |
| 830 | none | No. 9-7 | 17.00 | 10.15 | 42 | 344 | 320 | 79.3 | +12 | 63.8 | 18 |
| | | No. 11-7 | | | | 343 | 320 | 80.7 | +12 | 64.2 | 32.8 |
| | | No. 13-7 | | | | 343 | 319 | 80.0 | +11.5 | 64.2 | 20.3 |
| | | No. 15-7 | | | | 342 | 319 | 79.5 | +11 | 64.4 | 18 |
| | | Average | | | | 343 | 319.5 | 79.9 | +11.6 | 64.2 | 22.2 |
| | 150 | No. 9-8 | 17.05 | 10.40 | 38 | 351 | 324 | 82.3 | +15 | 62.9 | 32.8 |
| | | No. 11-8 | | | | 349 | 324 | 81.6 | +15 | 63.1 | 32.8 |
| | | No. 13-8 | | | | 350 | 323 | 81.7 | +14 | 63.1 | 21 |
| | | No. 15-8 | | | | 350 | 324 | 80.9 | +14 | 63.0 | 27.7 |
| | | Average | | | | 350 | 323.7 | 81.8 | +14.5 | 63.0 | 28.5 |
| | 180 | No. 9-9 | 17.13 | 10.64 | 38 | ... | ... | 83.6 | +18 | 62.2 | 31 |
| | | No. 11-9 | | | | 357 | 328 | 83.5 | +19 | 62.2 | 32.8 |
| | | No. 13-9 | | | | 356 | 327 | 83.6 | +18 | 62.2 | 33.5 |
| | | No. 15-9 | | | | [b] | [b] | [b] | [b] | [b] | [b] |
| | | Average | | | | 356.5 | 327.5 | 83.6 | +18.3 | 62.2 | 32.5 |
| | 210 | No. 9-10 | 17.47 | 11.06 | 32 | 370 | 335 | 85.4 | +25 | 60.7 | 36 |
| | | No. 11-10 | | | | 369 | 334 | 85.0 | +25 | 60.7 | 33.5 |
| | | No. 13-10 | | | | (361)[a] | (330)[a] | (82.6)[a] | (+10)[a] | 60.6 | 33.5 |
| | | No. 15-10 | | | | 367 | 334 | 84.2 | +24 | 60.8 | 34.9 |
| | | Average | | | | 369.6 | 334 | 84.8 | +24.6 | 60.7 | 34.8 |

[a] Short.

[b] Missing.

TABLE I.—*Continued.*

| Quenching Temperature, deg. Cent. | Tempering Temperature, deg. Cent. | Specimen | Tests by Sanford | | | Tests by Kouwenhoven | | Tests by de Forest | Tests by SKF Laboratories | Rockwell Hardness | Impact, kg-cm. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | $M_I$ | $R_I$ | $C_F$ | $\theta_L$ | $\theta_M$ | | | | |
| | | | | | | OIL QUENCHED | | | | | |
| 845 | none | No. 9-11 | 16.57 | 9.76 | 42 | 338 | 315 | 78.6 | +7 | 65.4 | 18.5 |
| | | No. 11-11 | | | | 338 | 315 | 78.7 | +7 | 65.1 | 21 |
| | | No. 13-11 | | | | 337 | 315 | 79.4 | +7 | 65.3 | 18 |
| | | No. 15-11 | | | | 337 | 314 | 78.5 | +6 | 65.6 | 20.3 |
| | | Average | | | | 337.5 | 314.7 | 78.8 | +6.7 | 65.4 | 19.5 |
| | 210 | No. 9-12 | 17.35 | 10.94 | 33 | 365 | 333 | 84.5 | +23 | 61.0 | 33.5 |
| | | No. 11-12 | | | | 366 | 333 | 84.9 | +23 | 61.4 | 33.5 |
| | | No. 13-12 | | | | 367 | 333 | 84.3 | +23 | 61.0 | 34.2 |
| | | No. 15-37 | | | | 366 | 332 | 83.5 | +22 | 61.5 | 33.5 |
| | | Average | | | | 366 | 332.7 | 84.3 | +22.7 | 61.2 | 33.6 |
| 860 | none | No. 9-13 | 16.48 | 9.63 | 45 | 336 | 314 | 78.4 | +5.5 | 65.8 | 17.3 |
| | | No. 11-13 | | | | [b] | [b] | [b] | [b] | [b] | [b] |
| | | No. 13-36 | 16.08 | 9.39 | 48 | 334 | 313 | 77.5 | +2 | 66.3 | 19.8 |
| | | No. 15-13 | | | | 335 | 314 | 79.2 | +6 | 66.1 | 17.3 |
| | | Average | 16.28 | 9.51 | 46.5 | 335 | 313.7 | 78.4 | +4.5 | 66.1 | 18.1 |
| | 150 | No. 9-14 | 16.57 | 9.92 | 40 | 345 | 320 | 79.6 | +10 | 64.3 | 22 |
| | | No. 11-14 | | | | 346 | 320 | 81.1 | +10 | 64.1 | 28.3 |
| | | No. 13-14 | | | | 345 | 320 | 79.9 | +10 | 64.4 | 21 |
| | | No. 15-14 | | | | 345 | 319 | 80.4 | +10 | 64.5 | 20.3 |
| | | Average | | | | 345.1 | 319.7 | 80.2 | +10 | 64.3 | 22.9 |
| | 180 | No. 9-15 | 16.85 | 10.30 | 39 | 355 | 327 | 81.8 | +16 | 63.0 | 34.1 |
| | | No. 11-15 | | | | 356 | 326 | 82.7 | +16 | 63.0 | 32.8 |
| | | No. 13-16 | | | | 355 | 326 | 82.1 | +16 | 63.1 | 32.8 |
| | | No. 15-15 | | | | 354 | 325 | 82.1 | +15 | 62.9 | 34.9 |
| | | Average | | | | 355 | 326 | 82.2 | +15.7 | 63.0 | 33.6 |
| | 210 | No. 9-16 | 16.97 | 10.56 | 38 | 366 | 332 | 84.0 | +21 | 61.4 | 38.8 |
| | | No. 11-35 | | | | 366 | 332 | 83.7 | +21 | 61.5 | 33.5 |
| | | No. 13-37 | 16.70 | 10.45 | 40 | 365 | 331 | 83.6 | +18 | 62.2 | 32.8 |
| | | No. 15-39 | | | | 365 | 331 | 83.7 | +21 | 61.7 | 34.9 |
| | | Average | 16.83 | 10.51 | 39 | 365.5 | 331.5 | 83.7 | +20.2 | 61.7 | 33.5 |
| 875 | none | No. 9-36 | 16.15 | 9.30 | 42 | 334.5 | 311 | 77.1 | +2 | 65.7 | 16 |
| | | No. 11-17 | 16.17 | 9.32 | 45 | 334.5 | 311 | 78.1 | +2 | 66.0 | 16.7 |
| | | No. 13-38 | 15.72 | 9.07 | 52 | 331 | 309 | .... | −2 | 66.2 | 17.3 |
| | | No. 15-17 | | | | 332 | 310 | 77.8 | +2 | 66.0 | 15.4 |
| | | Average | 16.01 | 9.23 | 46 | 333 | 310 | 77.7 | 1 | 65.9 | 16.3 |
| | 210 | No. 9-18 | 16.75 | 10.24 | 35 | 364 | 330 | 82.4 | +20 | 62.1 | 32.8 |
| | | No. 11-18 | | | | 364 | 329 | 83.6 | +20 | 62.1 | 32.8 |
| | | No. 13-39 | 16.33 | 10.09 | 39 | 362 | 328 | .... | +17 | 62.7 | 19.7 |
| | | No. 15-41 | 16.27 | 10.02 | 40 | 360 | 327 | 81.3 | +16 | 62.7 | 31 |
| | | Average | 16.45 | 10.12 | 38 | 363 | 329 | 82.4 | +18.2 | 62.4 | 29 |

[a] Short.
[b] Missing.

TABLE I.—*Continued.*

| Quenching Temperature, deg Cent. | Tempering Temperature, deg. Cent. | Specimen | Tests by Sanford | | | Tests by Kouwenhoven | | Tests by de Forest | Tests by SKF Laboratories | Rockwell Hardness | Impact, kg-cm. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | $M_1$ | $R_1$ | $C_F$ | $\theta_L$ | $\theta_M$ | | | | |
| | | | | | | WATER QUENCHED | | | | | |
| 800 | none | No. 9-19 | 17.65 | 10.84 | 38 | 357 | 326 | 81.4 | +17 | 65.2 | 19 |
| | | No. 11-19 | | | | 357 | 326 | 82.8 | +17 | 64.8 | 21 |
| | | No. 13-22 | | | | 356 | 326 | 82.1 | +17 | 64.9 | 21 |
| | | No. 15-19 | | | | 355 | 326 | 82 | +17 | 64.9 | 31.5 |
| | | Average | | | | 356 | 326 | 82.1 | +17 | 65.0 | 23.1 |
| | 150 | No. 9-20 | 17.58 | 11.03 | 33 | 363 | 332 | 84.8 | +20 | 62.9 | 34 |
| | | No. 11-20 | | | | 364 | 332 | 83.6 | +20 | 63.2 | 35 |
| | | No. 13-23 | | | | 363 | 331 | 83.4 | +21 | 63.1 | 26.5 |
| | | No. 15-20 | | | | 364 | 331 | 84.0 | +20 | 63.3 | 23 |
| | | Average | | | | 363.5 | 331.5 | 83.9 | +20.3 | 63.1 | 29.8 |
| | 180 | No. 9-21 | 17.75 | 11.2 | 32 | 370 | 335 | 84.2 | +24 | 61.9 | 33.5 |
| | | No. 11-21 | | | | 371 | 335 | 85.8 | +23 | 61.7 | 35 |
| | | No. 13-40 | 17.67 | 11.26 | 30 | 369 | 335 | 85.2 | +22 | 62.6 | 37 |
| | | No. 15-21 | | | | 369 | 334 | 83.9 | +23 | 62.0 | 34 |
| | | Average | 17.71 | 11.23 | 31 | 370 | 334.7 | 85.0 | +23 | 62.1 | 34.9 |
| | 210 | No. 9-22 | 17.95 | 11.50 | 28 | 375 | 339 | 86.5 | +30 | 60.0 | 37.5 |
| | | No. 11-22 | | | | 377 | 338 | 86.6 | +29 | 60.2 | 37.5 |
| | | No. 13-25 | | | | 375 | 337 | 85.8 | +28 | 60.5 | 41.5 |
| | | No. 15-46 | 17.90 | 11.65 | 30 | 375 | 338 | 85.5 | +27 | 61.0 | 39.5 |
| | | Average | 17.93 | 11.58 | 29 | 375.5 | 338 | 86.1 | +28.5 | 60.4 | 39 |
| 815 | none | No. 9-23 | 17.55 | 10.70 | 39 | 353 | 325 | 81.1 | +15 | 65.6 | 21 |
| | | No. 11-23 | | | | 354 | 325 | 82.2 | +15.5 | 65.3 | 29.5 |
| | | No. 13-26 | | | | 354 | 324 | 81.7 | +15 | 65.4 | 24 |
| | | No. 15-42 | 17.37 | 10.72 | 42 | 353 | 322 | 81.5 | +12 | 65.7 | 31.5 |
| | | Average | 17.46 | 10.71 | 40 | 353.5 | 324 | 81.8 | +14.4 | 65.4 | 26.5 |
| | 210 | No. 9-24 | 17.85 | 11.44 | 30 | 378 | 338 | 86.2 | +28 | 60.7 | 35.5 |
| | | No. 11-24 | | | | 378 | 338 | 85.7 | +29 | 60.6 | 36.2 |
| | | No. 13-41 | | | | 377 | 337 | 84.8 | +27 | 60.9 | 36.2 |
| | | No. 15-24 | | | | 377 | 338 | 85.5 | +28 | 60.4 | 41.5 |
| | | Average | | | | 377.5 | 337.7 | 85.5 | +28 | 60.7 | 37.4 |
| 830 | none | No. 9-25 | 17.35 | 10.50 | 39 | 353 | 322 | 81.0 | +13 | 64.9 | 18.5 |
| | | No. 11-25 | | | | 354 | 322 | 81.3 | +13 | 65.9 | 20 |
| | | No. 13-28 | | | | 352 | 322 | 80.2 | +13 | 66.0 | 18.5 |
| | | No. 15-25 | | | | 351 | 322 | 81.8 | +13 | 66.0 | 18 |
| | | Average | | | | 352.5 | 322 | 81.1 | +13 | 65.7 | 19 |
| | 150 | No. 9-26 | 17.43 | 10.82 | 37 | 362 | 329 | 82.9 | +18 | 64.1 | 19.2 |
| | | No. 11-26 | | | | 362 | 329 | 84.2 | +18 | 63.9 | 23 |
| | | No. 13-29 | | | | 361 | 328 | 82.8 | +19 | 64.2 | 21 |
| | | No. 15-26 | | | | 361 | 328 | 83.6 | +18 | 64.6 | 20.5 |
| | | Average | | | | 361.5 | 328.5 | 83.1 | +18.2 | 64.2 | 21 |
| | 180 | No. 9-27 | 17.55 | 11.06 | 30 | 370 | 334 | 85.0 | +22 | 62.6 | 33.5 |
| | | No. 11-27 | | | | 369 | 333 | 84.9 | +22 | 62.5 | 33.5 |
| | | No. 13-42 | 17.40 | 10.99 | 32 | 369 | 333 | 83.8 | +21 | 62.7 | 33.5 |
| | | No. 15-43 | 17.37 | 11.06 | 32 | 367 | 332 | .... | +20 | 63.7 | 33.5 |
| | | Average | 17.44 | 11.04 | 31.3 | 368.7 | 333 | 84.6 | +21.3 | 62.9 | 33.5 |
| | 210 | No. 9-28 | 17.77 | 11.36 | 30 | 380 | 339 | .... | +28 | 60.6 | 34 |
| | | No. 11-37 | 17.55 | 11.30 | 32 | 377 | 336 | 84.5 | +25 | 61.7 | 35 |
| | | No. 13-31 | | | | 379 | 338 | 86.0 | +27 | 61.5 | 36.2 |
| | | No. 15-28 | | | | 378 | 337 | 86.3 | +25 | 61.0 | 36.2 |
| | | Average | 17.66 | 11.33 | 31 | 378.5 | 337.5 | 85.6 | +27.7 | 61.2 | 35.4 |
| 845 | none | No. 9-29 | 17.33 | 10.48 | 39 | 350 | 320 | 79.8 | +12 | 66.0 | 23.5 |
| | | No. 11-29 | | | | 351.5 | 321 | 81.5 | +12 | 66.4 | 17.4 |
| | | No. 13-43 | 17.00 | 10.31 | 43 | 350 | 319 | 79.5 | +8 | 66.3 | 17.4 |
| | | No. 15-44 | 16.97 | 10.28 | 49 | 349 | 318 | 79.9 | +8 | 66.1 | 15 |
| | | Average | 17.10 | 10.36 | 43 | 350 | 319.5 | 80.2 | +10 | 66.2 | 18.6 |
| | 210 | No. 9-30 | 17.65 | 11.36 | 33 | 376 | 337 | 85.6 | +27 | 61.7 | 33.5 |
| | | No. 11-30 | | | | 377 | 337 | 85.7 | +27 | 61.7 | 34 |
| | | No. 13-44 | 17.37 | 11.08 | 35 | 375 | 336 | 84.5 | +24 | 62.4 | 33 |
| | | No. 15-45 | 17.43 | 11.18 | 32 | 375 | 335 | 84.2 | +25 | 62.2 | 33.5 |
| | | Average | 17.48 | 11.20 | 33 | 375.7 | 336 | 85.0 | +25.7 | 62.0 | 33.5 |

results of the mechanical tests before attempting a more detailed discussion of the data.

Mr. A. V. de Forest made the tests on his inductance bridge using the null method and has plotted his average readings as shown in Fig. 2. The individual readings are given in Table I.

Mr. W. B. Kouwenhoven, using a method described before the Society in 1930,[1] submitted his results in form of charts which are shown slightly rearranged in Fig. 3.

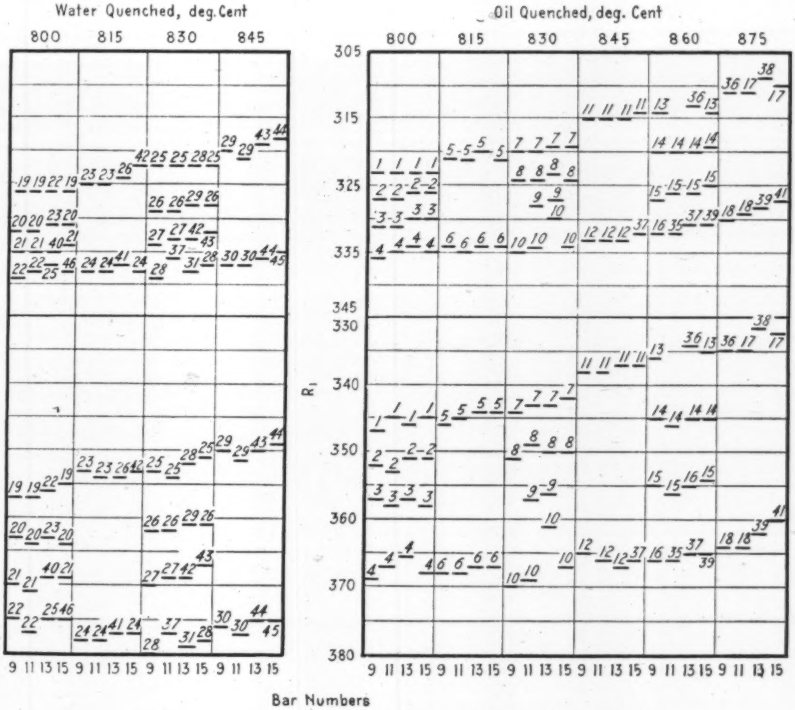

FIG. 3.—Showing Results Obtained by W. B. Kouwenhoven.

He states:

These charts give the data obtained in testing these specimens in a four-arm impedance bridge using alternating-current 60-cycle supply. Two solenoids were used, one containing the standard, specimen No. 9–9, and the other containing the specimen to be tested. The ordinates used in the chart give the value of the variable resistance arm $R_1$ of the bridge for balance. Two measurements were made with each bar, one proportional to the magnetizing component and the other to the loss. $\theta_M$ expresses the magnetizing components and $\theta_L$ the loss.

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[1] W. B. Kouwenhoven and A. C. Seletzky, "The Unbalanced Alternating Current Bridge for Magnetic Analysis," *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part II, p. 298 (1930).

I consider that the bars are remarkably uniform in their heat treatment because the specimens having the same treatment had very nearly the same value of $R_1$ for balance. These tests were performed at a magnetizing force of roughly 100 gilberts per centimeter in an open magnetic circuit. Bars 13–10 and 13–4 were of a length differing from the standard bar and therefore are displaced in their location on the charts.

Magnetic tests were also made at the SKF Research Laboratory, first, on all specimens before hardening, for control of uniformity and second on all specimens after quenching and grinding to final dimension, before tempering, and again on all tempered specimens.

All individual magnetic readings are given in Table I together with the average for each group of heat treatment. After these tests, the specimens were notched with a 2-mm. emery wheel to the dimensions given in Fig. 1. Before the final depth was reached, the edge of the grinding wheel was dressed to a radius of 1 mm. and the slot carefully finished. Etching with 5-per-cent nital was used to insure that no tempering occurred in this grinding.

Finally, impact tests were made on a 10-kg-m. Charpy impact machine made by A. B. Atlas, Stockholm, and four Rockwell impressions were made on one-half of each specimen at the SKF Research Laboratory.

The Rockwell machine was standardized against a Bureau of Standards specimen having a Rockwell hardness of 60 and a Rockwell sample having a hardness of 66.5.

In Rockwell testing, soft spots from surface troostite on some of the water-quenched specimens were avoided, as otherwise those readings would give a false impression of the average structure.

The impact and Rockwell values of each specimen are also given in the table, together with the group average.

Rockwell hardness tests on the other half of the specimens were made at the Republic Research Corporation by Mr. Grossman. His values as an average of two readings for the untempered specimens and specimens tempered at 210° C., are shown in Fig. 4. His relative values show the same trend as the SKF values, although they do not average the same. He took no precaution to avoid surface troostite on water-quenched specimens. Intercomparison of his Rockwell readings with SKF was unfortunately impossible due to later removal of his Rockwell apparatus. The average readings for all tests are plotted for comparison in Fig. 4, in which the scale for each method of investigation is chosen to give approximately the same total spread. The solid lines represent untempered specimens, the broken lines specimens tempered at 210° C. Specimens tempered at 150° C. and those tempered at 180° C., are appropriately designated.

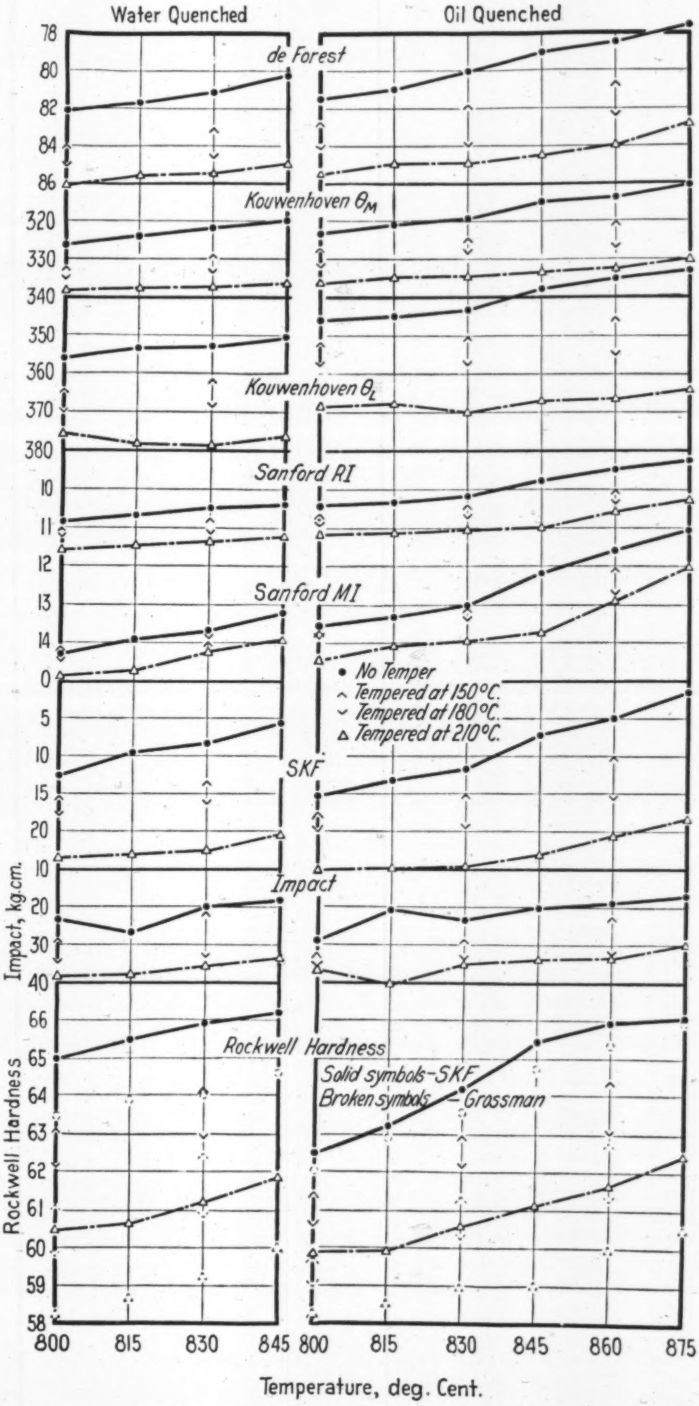

FIG. 4.—Showing Comparison of Magnetic Test Results with Rockwell Hardness and Impact Values.

The coercive force is not shown but has the same definite trend as the other curves for quenching temperatures over 830° C.

## DISCUSSION

The individual deviation from the average for each heat treatment is about the same for each method of magnetic test.

Variation in length of specimens would have been very noticeable in all magnetic methods, except Sanford's.

Variation in cross-section would also be very noticeable in de Forest and SKF test.

The impact values naturally show a greater individual deviation although the average trend is very distinct. The Rockwell values made at SKF show less individual deviation, but here the individual values again are averages of four readings. Values obtained by two different operators would, however, probably vary more. The average of all the magnetic readings of untempered specimens would give a smoother curve for SKF readings than the group values given in Fig. 4. A method of very great sensitivity such as, for instance, the de Forest method compared with the less sensitive SKF method gives about the same percentage deviation.

## CONCLUSIONS

Four different magnetic methods of testing give about the same correlation to hardness and impact strength of a simple alloy steel hardened over a wide range.

Maximum induction and residual induction decrease with higher hardening temperature and increase with higher tempering temperature. Both are lower for oil quenched than for water-quenched material. The coercive force is more irregular but there is an apparent increase with higher hardening temperatures and decrease with higher tempering temperatures.

Hardness increases with higher hardening temperatures and is higher for water-quenching than for oil-quenching at corresponding temperatures, and drops, of course, with higher tempering temperatures. Impact strength decreases with higher hardening temperatures and increases with higher tempering temperatures. Water and oil quenching give about the same values for impact strength. An increase in hardness corresponds to a decrease in impact strength.

For properly heat-treated material of controlled dimensions, the individual variation in relative magnetic readings is well within a range of variation that is found by usually applied mechanical tests such as hardness and impact tests, and have the great advantage that they are non-destructive and reproducible.

With properly selected standards of uniform section and with suitable sensitivity of apparatus, which can be varied practically at will, the uniformity of any heat treatment within commercial temperature tolerances can be controlled by magnetic methods with a reliability and accuracy equal to or better than those obtaining with the usual mechanical methods of control. Within such temperature ranges as used and for the same kind of heat treatment, there is a fair degree of proportionality between magnetic tests and certain mechanical tests.

It is likely that magnetic methods which are independent of form and size could be developed to give equally satisfactory results over a reasonable range of heat treatment.

# DISCUSSION

MR. HOWARD SCOTT.[1]—In the paper by Mr. Styri, I was particularly interested in the fact that he finds what he calls quenching troostite, I believe, on the surface of the specimen quenched in water. That is a very interesting observation, and I wonder if Mr. Styri has any explanation of it?

MR. HAAKON STYRI.[2]—Formation of surface troostite is a common phenomenon in quenching steel in water, and it is probably due to steam bubbles forming on the surface; but an additional factor is that these specimens were heated in a salt bath and had a skin of salt on the surface. We had intended to quench those specimens in brine, and would then probably not have gotten the surface troostite. There was no surface troostite on the oil-quenched samples; the amount present on the water-quenched specimens was so small we could hardly detect anything in the magnetic tests, but it was observed in the Rockwell hardness test. However, the amount was not sufficient to affect the individual Rockwell reading more than perhaps half a point or one point, and the average variation would be less. The values given in the paper were taken on areas which did not show surface troostite.

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[1] Section Engineer, Research Laboratories, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[2] Director of Research, SKF Industries, Inc., Philadelphia, Pa.

# THE UNBALANCED ALTERNATING CURRENT BRIDGE FOR MAGNETIC ANALYSIS II

By W. B. Kouwenhoven[1] and J. H. Lampe[2]

## Synopsis

This paper presents a discussion of improvements in the unbalanced alternating-current bridge developed for the magnetic testing of specimens differing in cross-sectional area. The results show clearly that the improved bridge provides a simple means of separation of specimens of unequal cross-section into definite groups according to heat treatment.

The operation of the bridge was studied at various frequencies and it is definitely shown that flux penetration plays a part in its operation at 60 cycles. The paper further describes the investigations made with a view to increasing the sensitivity of the bridge. Data are given as to the best types of magnetizing solenoids to be used, the method of determining the most satisfactory values of the magnetomotive force, the proper values of the impedance of the arms of the bridge, and the improved operation obtained by using a constant alternating voltage across the bridge.

Finally, the results of a series of tests on knife blades are given and compared with those obtained with the bridge as originally developed. This comparison shows clearly very large increases in separation of specimens when using the improved bridge arrangement. The increased and definite separation of specimens makes possible a good correlation between their magnetic characteristics as measured in the bridge, and their Brinell hardness.

## Introduction

Of all the methods of magnetic analysis for testing steel as to its quality or degree of heat treatment, the alternating-current methods seem to offer the best possibilities. These methods usually use a moving coil alternating-current galvanometer as a detector. The magnetic field of this type of galvanometer is set up by a separate field winding which carries current of the same frequency as that used in testing the specimens. By varying the phase of the current through the galvanometer field the deflection of the instrument may be made proportional to the magnetic flux in the specimen, the losses, or to any desired combination of these two.*

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[2] Associate in Electrical Engineering, School of Engineering, Johns Hopkins University, Homewood, Baltimore, Md.

One of the authors described an alternating-current method in a paper presented before the Society last year.[1] In this method an unbalanced alternating-current bridge was used and it was discovered that at a certain phase setting of the galvanometer field, the readings of the bridge were independent of the cross-sectional area of the specimens tested. The present paper deals with a further study of the unbalanced alternating current bridge with reference to the best type of test coil, the effect of variations in bridge arrangements, and other methods of increasing the sensitivity. This study has led to a bridge which possesses high sensitivity and which gives a large and definite separation between specimens differing slightly in quality.

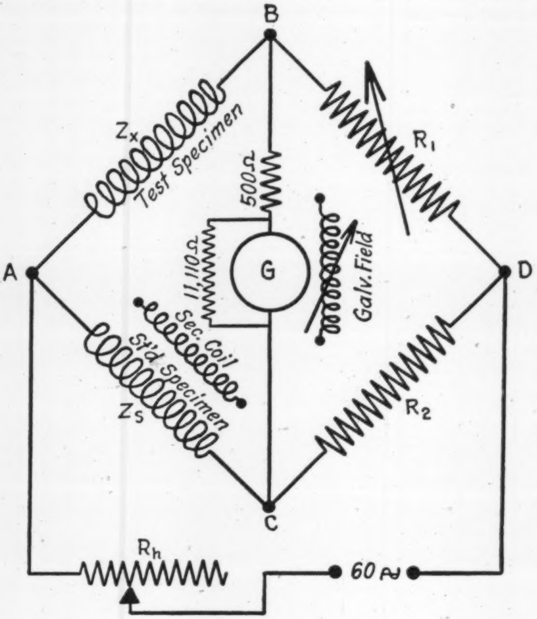

FIG. 1.—Diagram of Bridge Circuit.

## THE BRIDGE

A brief description of the unbalanced alternating-current bridge is repeated here to familiarize the reader with its operation. Complete details may be obtained by referring to the previous paper.[1]

The bridge is shown in Fig. 1. The two coils for magnetizing the specimens are marked $Z_x$ and $Z_s$ respectively. $R_2$ is a fixed resistance, and $R_1$ a variable resistance of the slide wire type. The galvanometer used was a Leeds & Northrup pointer type alternating-current galvanometer, with a separately excited field winding.

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[1] W. B. Kouwenhoven and A. C. Seletzky, "The Unbalanced Alternating Current Bridge for Magnetic Analysis," *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part II, p. 298 (1930).

Two specimens alike in quality and area are placed, one in each coil, and the voltage across the bridge terminals adjusted to give the desired current through the coil containing the standard specimen. The bridge is then balanced by adjusting the variable resistance $R_1$. Under this condition the bridge balance is independent of the phase of the galvanometer field. The full size specimen is now withdrawn from the test coil, and another specimen of the same magnetic properties and length, but of different area is inserted in its place. The resulting galvanometer deflection is then reduced to zero by varying the phase setting of the galvanometer field. This setting of the galvanometer field is called $\theta_x$, and is a function both of the flux and of the loss in the specimen.

The bridge is now ready for operation. The voltage across it is kept constant and the specimens to be tested are inserted in turn in the test coil. Any variation in the value of $R_1$ for balance indicates that the unknown specimen differs from the standard.

The use of the standard specimen may be omitted, and a coil alone used in its place, provided other means are available for balancing the bridge. This avoids the possibility of heating of the standard specimen and the consequent changes that may take place in its characteristics. However, by proper design of the magnetizing winding a standard specimen may be safely used.

In addition to the $\theta_x$ reading, it is possible to obtain a different setting of $R_1$ for balance for each position of galvanometer field. The two most important observations are, first, the flux setting $\theta_M$, where the change in the value $R_1$ is a measure of the difference in flux in the two specimens, and second, the loss setting $\theta_L$, where the change in $R_1$ is a measure of the difference in the losses. Readings obtained at these two settings proved to be of great value in the study of the magnetic properties and heat treatment of the specimens.

## FLUX PENETRATION

Magnetic analysis by the use of alternating flux involves two factors, permeability and losses. In addition, the penetration of the flux into the material must be considered. This phenomena, known as the "skin effect" was suggested by L. W. McKeehan[1] as a possible explanation of the operation of the unbalanced alternating-current bridge, with the galvanometer field in the $\theta_x$ position.

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[1] Discussion by L. W. McKeehan of paper by W. B. Kouwenhoven and A. C. Seletzky on "The Unbalanced Alternating Current Bridge for Magnetic Analysis," *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part II, p. 312 (1930).

When a solid steel specimen is subjected to an alternating magnetizing force, the flux sets up eddy currents in the material. The action of the eddy currents cause a weakening of the alternating-current flux. This action will be a maximum at the center of the specimen and results in a concentration of the magnetic flux toward the outer layer. The flux density throughout the specimen is, therefore, not uniform. Skin effect is usually expressed as the penetration of the flux at surface density into the specimen, which gives the same total flux as actually exists.

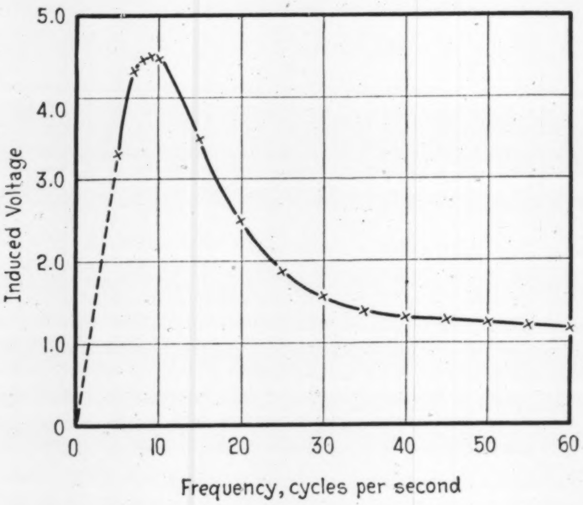

FIG. 2.—Flux Penetration Phenomena, Magnetizing Force Constant.

The distance of penetration of flux into a steel specimen is given by the well-known fundamental equation:

$$\sigma = K\sqrt{\frac{\rho}{\mu f}}$$

where $\rho$ = specific resistance of specimen,
$\sigma$ = penetration in centimeters,
$K$ = constant,
$\mu$ = permeability of specimen, and
$f$ = frequency.

It is seen from these relations that the total thickness is not a contributing factor with regard to total flux present. On the other hand, the thinner the specimen, the greater will be the relative penetration and the more uniform the flux density throughout. With a given thickness of specimen, the penetration will remain the same, with constant frequency independent of cross-sectional area.

In order to determine if the phenomena of "skin effect" was present to a marked degree in the knife blade specimens used in this investigation, their magnetic characteristics were studied at a constant magnetizing force of variable frequency. Specimens of the same stock, thickness and heat treatment as the knife blades were tested in the laminated Fahy Simplex permeameter.

The voltage induced in the "*B*" coil of the permeameter was measured at a constant magnetizing force for a wide range of frequencies. Assuming that the permeability is independent of frequency as has been shown by Alexanderson[1] and others, the induced voltage would be directly proportional to the frequency if no skin effect were present.

Results of this work are shown in Fig. 2. It is evident that the flux penetration is uniform up to a frequency of approximately seven cycles per second for the condition of the test. The induced voltage reaches a maximum at nine cycles. Beyond this point the total flux threading the specimen decreases as the frequency is increased. This curve also shows that at the higher frequencies the net flux is approximately constant.

This curve (Fig. 2) shows definite evidence of the phenomenon of skin effect in the steel knife blade specimens which have a maximum thickness of 0.1 in. The results, therefore, indicate that flux penetration is a factor in the operation of the unbalanced alternating-current bridge with specimens differing in cross-sectional area.

## DEVELOPMENT

In addition to the knife blades mentioned in the previous paper, another group of blades was studied. This group of blades, known as lot C, was submitted by the Winchester Repeating Arms Co. and consisted of 26 unknown blades marked A to Z, inclusive, and a group of seven standard blades marked S-1 to S-7, inclusive. The 26 unknown blades included some with softened points and some others with slight differences in heat treatment. These blades were tested in the original bridge and their separation was so slight that no data concerning the correlation between their magnetic characteristics and heat treatment were published.

The authors were not satisfied with the results obtained and, therefore, studied the bridge and its operation further. The outcome of this study led to a very much improved segregation of the 26 unknown blades into their respective groups and shows definite correlation between bridge readings and heat treatment.

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[1] E. F. W. Alexanderson, "Magnetic Properties of Iron at Frequencies up to 200,000 Cycles," *Transactions*, Am. Inst. Electrical Engrs., Vol. 30, Part III, p. 2433 (1911).

The first improvement was obtained by operating the bridge at constant voltage instead of constant current, as suggested in the first paper. The next step was to obtain the complete circle diagram of the bridge, experimentally. This is given in the Appendix to this paper.

The bridge was then studied as a four arm inductance bridge and the values of the impedance in its arms brought into agreement with the conditions for maximum sensitivity. This study also brought to light certain relations that applied to the impedance of the bridge arms. Next the effect of wide variations in magnetomotive force upon the separation of the specimens was investigated. Finally the results of these studies were combined and produced an improved unbalanced alternating-current bridge, which gives definite indications of variation in heat treatment of specimens, independent of their cross-sectional area.

## CONSTANT VOLTAGE OPERATION

The unbalanced alternating-current bridge as previously presented used a constant current through the specimen test coil for specimens of all sizes and permitted the current to vary through the standard test coil. This method, while working satisfactorily, presents the practical difficulty that the bridge voltage varies for each specimen. This fact would be undesirable in commercial application. After an extensive investigation it was found that the unbalanced alternating-current bridge with some minor changes separated steel specimens of various heat treatments to an even better degree when the voltage was maintained constant across the entire bridge. This method of operation maintains the current through the standard test coil constant, while the current through the specimen test coil now varies as the specimen deviates from the standard.

When the voltage is held constant across the bridge, the voltages and currents in the bridge vary in accordance with the circle diagram relations of a series circuit containing resistance and reactance. The values of the voltages and currents in the bridge arms were determined for full, one-half, and quarter size specimens and are plotted in the circle diagram of the bridge. This diagram is shown in Fig. 4 in the Appendix.

## IMPEDANCE OF BRIDGE ARMS

A study of the circle diagram of the alternating-current bridge shows the voltage across the test coil to be very much greater in value than the voltage on the resistance arms of the bridge. It is a well-known fact that if the voltages on the four arms of the bridge are of the same value, the sensitivity of the combination will be a maximum. To accomplish this either the impedance of the test coils must be decreased, or the value of resistance in the resistance arms of the bridge must be increased.

In an endeavor to decrease the resistance of the test coils, new coils of 720 turns of No. 16 D.C.C. wire were built. The resistance of each coil was 1.5 ohms (direct current) and the inductance 7.3 millihenries without the specimen in place, and 10.6 millihenries with the standard specimen. These coils, which were designated as coils Nos. 3 and 4, respectively, had a lower impedance than the ones used in the previous work. The lower impedance of these coils gave voltages of approximately the same order of magnitude on all arms, but the sensitivity of the bridge was decreased due to the large leakage flux and the low effective magnetizing force. As the resistance of a coil is decreased, its leakage flux increases due to the larger size wire needed. Results mentioned in a later part of this paper show that a large magnetomotive force is necessary if increased separation of specimens is to be obtained.

To increase the flux density in the knife blade specimens, another set of coils was constructed, each of 1920 turns of No. 24 D.C.C. wire. These coils, Nos. 5 and 6 respectively, each had a resistance of 18.2 ohms (direct current) and an inductance of 32.6 millihenries without the specimen, and 83.8 millihenries with the standard specimen in place. The resistances of the $R_1$ and $R_2$ arms of the bridge were increased until the voltages across the four arms were of the same order of magnitude. Increased sensitivity and separation was evident at once for this circuit as compared to other bridge arrangements.

The high value of resistance of test coils Nos. 5 and 6 might first be considered as a detriment to the operation of this method, but such is not the case. It must be remembered that this alternating-current bridge measures the difference in the losses between specimens in the two test coils, and when both coils contain similar specimens in every respect, all the losses in one side of the bridge are balanced by those in the other side. When specimens of different characteristics are tested, the bridge reading is a measure of their difference.

After constructing the coils they were adjusted so as to give equal magnetizing forces for the same current values and were also made equal in resistance. After the test coils were thus balanced the bridge readings were independent of the phase of the galvanometer field for a specimen which had indentical characteristics and dimensions with those of the standard.

## MAGNETOMOTIVE FORCE

The value of the magnetomotive force that will give the best separation of specimens varies with the material. Using test coils Nos. 5 and 6 and two knife blade specimens S-1 and A, the effect of varying the exciting current was studied. The result of this test showed that the best operating conditions were obtained when a current of 0.6 ampere flowed through the 1920 turns of the solenoid. This gave the largest separation between the specimens, that is, the maximum difference in their $R_1$ settings for balance.

TABLE I.—RESULTS OF MAGNETIC ANALYSIS TESTS ON KNIFE BLADES.

| Blade | Brinell Hardness Numbers | | | $\theta_x$ Separation, $R_1$ Reading | Blade | Brinell Hardness Numbers | | | $\theta_x$ Separation, $R_1$ Reading |
|---|---|---|---|---|---|---|---|---|---|
| | Heel | Middle | Point | | | Heel | Middle | Point | |
| STANDARD GROUP: DRAWN AT 450° F. FOR 30 MINUTES. | | | | | | | | | |
| S-1 | 592 | 623 | 592 | .... | S-6 | 592 | 623 | 592 | .... |
| S-2 | 592 | 592 | 592 | .... | S-7 | 592 | 592 | 592 | .... |
| S-5 | 572 | 592 | 572 | .... | | | | | |
| GROUP I: STANDARD HEAT TREATMENT. | | | | | | | | | |
| D | 572 | 592 | 592 | 5 | U | 572 | 592 | 572 | −9 |
| V | 592 | 623 | 623 | 7 | X | 592 | 623 | 592 | −10 |
| I | 572 | 592 | 592 | 0 | G | ... | ... | ... | 5 |
| GROUP II: STANDARD HEAT TREATMENT. POINTS OF BLADES THEN HEATED FOR ½ IN. TO A RED COLOR IN A LEAD BATH AND COOLED IN AIR. | | | | | | | | | |
| S | 592 | 592 | 355 | −39 | M | 592 | 592 | 408 | −30 |
| R | 592 | 592 | 396 | −28 | N | 592 | 592 | 341 | −27 |
| J | 623 | 592 | 396 | −44 | | | | | |
| GROUP III: DRAWN AT 950° F. FOR 30 MINUTES. | | | | | | | | | |
| A | 396 | 396 | 396 | 169 | W | 396 | 396 | 396 | 173 |
| B | 396 | 408 | 396 | 167 | Z | 396 | 408 | 408 | 170 |
| K | 408 | 408 | 408 | 169 | | | | | |
| GROUP IV: DRAWN AT 850° F. FOR 30 MINUTES. | | | | | | | | | |
| C | 426 | 426 | 426 | 137 | P | 426 | 426 | 426 | 145 |
| F | 426 | 426 | 426 | 146 | Y | 426 | 426 | 426 | 141 |
| L | 426 | 426 | 426 | 143 | | | | | |
| GROUP V: DRAWN AT 650° F. FOR 30 MINUTES. | | | | | | | | | |
| E | 519 | 519 | 519 | 59 | Q | 503 | 519 | 503 | 45 |
| H | 519 | 519 | 519 | 61 | T | 519 | 503 | 503 | 55 |
| O | 503 | 519 | 519 | 48 | | | | | |

## RESULTS

The results of the described investigations all point to means of improved sensitivity of the bridge and increased separation between specimens, and therefore it was only necessary to take advantage of these improvements and construct a new alternating-current birdge. This was done, using coils of improved design to give greater flux density in the specimen. Constant voltage was used across the bridge in order to obtain better operation and further, the values of the bridge arms were so adjusted as to give voltages of the same order of magnitude across all of them. Preliminary tests showed large increases in separation of steel speimens over those obtained in the first work published on this subject.

Many tests were conducted on this bridge, and a characteristic group of data is shown for one complete run on the knife blade specimens mentioned above. They were in the rough state before grinding and differed slightly in cross-sectional area. The blades were stamped on the tang, alphabetically from A to Z, and had received various heat treatments. In addition to this lot of blades, was a group of seven blades stamped S-1 to S-7, which had received the standard heat treatment and had the standard degree of hardness. Specimen S-1 was used as a standard and specimen S-3 was reduced to approximately one half cross-sectional area. The latter specimen was used in establishing the galvanometer phase position $\theta_x$. These knife blades had approximately the same dimensions as those mentioned in the previous paper. For this test the bridge was operated at 35 volts, 60 cycles.

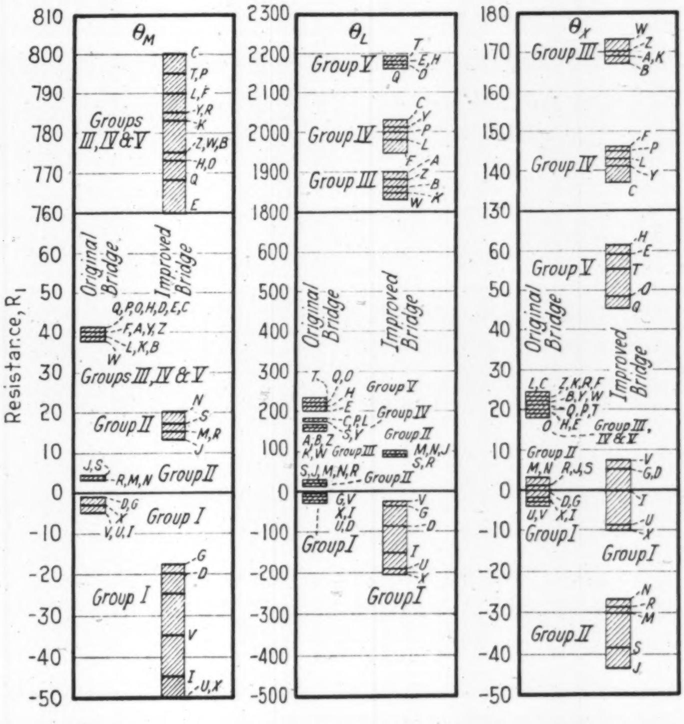

FIG. 3.—Separation of Specimens of Varying Heat Treatment.
Specimen S-1 Used as Standard.

The heat treatment, Brinell hardness number at three locations, and the bridge reading at the $\theta_x$ setting of each specimen are given in Table I. The knife blade stock had been quenched at 1550° F. before receiving the heat treatment indicated. The blades fall into five groups, as may be seen from Table I.

The results are given in Fig. 3 of the bridge readings obtained as the flux setting, $\theta_M$, at the loss setting, $\theta_L$, and at the combination setting $\theta_x$, of the galvanometer field. In order to show clearly the improvements that have been realized, the values obtained with the original bridge a year ago are included in the figure. A study of this figure shows that the separation between the specimens of different heat treatments has been increased at the $\theta_M$ galvanometer setting roughly twenty times, at $\theta_L$ about ten times, and at $\theta_x$ some eight times.

## SUMMARY

It is evident that skin effect is present in the knife blade specimens and its action contributes to the successful operation of the bridge when the galvanometer field setting is at the combination position, $\theta_x$.

Greatly increased sensitivity is made possible by the use of the proper values of resistances and inductances in the bridge arms. The resistance and inductance of the standard coil must balance those of the unknown coil.

For any given specimens the best value of the magnetomotive force may be easily determined by test. In winding magnetizing solenoids for use in an alternating-current bridge, the resistance may be any value consistent with safe heating, because the bridge reading is a measure of the difference between specimens.

The bridge in its improved form not only separates the good specimens from those that have received improper treatment but also divides them into distinct groups, in line with both their heat treatments and Brinell hardness numbers.

## APPENDIX

### *Circle Diagram:*

The circle diagram, Fig. 4, was obtained from measurements of the voltages and currents in the original bridge using specimens differing in area, but of identical magnetic characteristics. The lettering on the vector diagram is the same as that of Fig. 1. In Fig. 4, *AD* is the voltage across the bridge. The vector *AC* is the drop across the standard coil, $Z_2$; *CD* the voltage drop across the resistance $R_2$.

With identical specimens in the two coils, the voltage drop *AB*, across the unknown, coincides with *AC*; and also *BD*, the drop across $R_1$, falls on *CD*.

That is, the points $C$ and $B$ coincide when the bridge is perfectly balanced When the unknown specimen has a cross-section different from that of the standard, the point $B$ no longer coincides with $C$. The new location of the point $B$ is shown by the crosses for a one-half size specimen in the $Z_x$ coil for each of the three positions $\theta_M$, $\theta_L$ and $\theta_x$ of the galvanometer field. The points shown by the circles are the corresponding positions for a one-quarter size specimen. It is evident that with specimens of unequal areas, the point $B$ must be brought to its proper location for zero deflection of the galvanometer at the given phase setting by adjusting the variable resistance $R_1$.

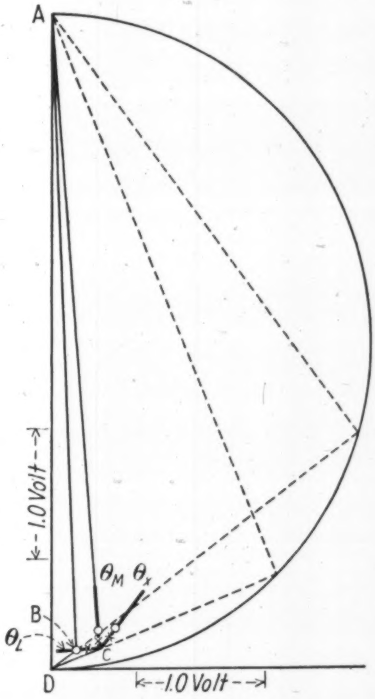

FIG. 4.—Circle Diagram of Bridge Circuit, Applied Voltage Constant.

# THE MEASUREMENT OF LARGE BRINELL IMPRESSIONS IN STEEL RAILS

By H. H. Morgan[1] and J. R. Mooney[2]

## Synopsis

In the manufacture of both tee and girder rails, an indentation test is made on the head of test rails with a 19-mm. (¾-in.) ball and 100,000-lb. load. For the measurement of these impressions, the Hunt Brinell gage has been developed which indicates the same hardness as is obtained from standard size Brinell impressions made on a machined surface. Simple measurements of either depth or diameter of the large impressions do not indicate the correct hardness.

## Introduction

In the manufacture of both tee and girder rails, an indentation test is made on the head of test rails with a 19-mm. (¾-in.) ball and 100,000-lb. load. The results of these indentation tests are the basis of acceptance or rejection of entire heats in the case of girder rails, and although they are reported as information only in the case of tee rails, they are important in consideration of rail failures and of the wear obtained from the rails.

The inspectors of the Robert W. Hunt Co. witnessing these tests, personally obtain and record all test data. The accurate measurement of these impressions on the rough and curved surfaces of rails by many different men under the conditions of dirt and poor light prevailing at the mills was so difficult that a special instrument was developed for this work.

In making these tests on steel rails, the curvature and roughness of the head and the scale necessitate the use of a larger ball than the usual ⅜-in. size. A 19-mm. (0.748-in.) ball under a 100,000-lb. load is used, and makes an impression about 0.1 in. deep by 0.5 in. in diameter with a ridge or bulge around the edge about 0.01 in. higher than the original surface. (See Fig. 1.) This ridge slopes away to the level of the original surface in about one inch. Experience has shown that this impression is large enough so that the surface irregularities do not seriously affect the accuracy of the test.

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Brinell hardness is defined as the ratio of the load to the spherical area of the impression. This spherical area cannot be measured directly but must be calculated either from the diameter or the depth of the impression, but because the impression is not perfectly spherical (being shallower than a true section of a sphere) and has an imperfectly defined edge with a ridge around it, various values of depth and diameter may be obtained, as shown in Fig. 1, all giving different hardness values. Since the object of the test is to determine the Brinell hardness of the material, the dimension which most accurately and uniformly gives this hardness is the proper dimension to use.[1]

## PRELIMINARY EXPERIMENT

The head of a new steel rail is covered with mill scale and the metal decarburized for an appreciable depth, though usually not exceeding $\frac{1}{64}$ in. To determine what effect the scale and decarburized metal has, there were made on the head of a rail, three Brinell impressions, using a $\frac{3}{8}$-in. ball and 3000-kg. load, without any preparation of the surface. Adjacent to these impressions, 0.03 in. was machined from the head, three impressions made, and then at different locations, 0.5 in. of metal was machined from the head and three more tests made. The average hardness indicated by the tests made on the original surface was 249, the hardness 0.03 in. below the surface 256, and 0.5 in. below the surface, 259. It is evident that there is little change of hardness below 0.03 in., and all subsequent standard size tests on rails were made on a machined surface 0.05 in. below the top of the head, using a 50,000-lb. testing machine at $\frac{1}{8}$ in. per minute up to near the maximum load and finishing with the hand wheel. The load was maintained for about fifteen seconds.

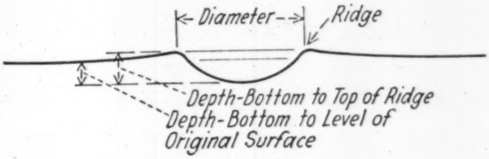
Diameter
Ridge
Depth-Bottom to Top of Ridge
Depth-Bottom to Level of Original Surface

FIG. 1.—Diagrammatic Sketch of Cross-Section of Large Brinell Impression.

## DETERMINATION OF PROPER DIMENSION OF LARGE IMPRESSION TO BE MEASURED

Eight rails were selected so that both ordinary carbon steel and intermediate manganese steel were represented, and on them were made three impression tests with a 19-mm. ball and 100,000-lb. load. These were made with a 300,000-lb. testing machine at ½ in. per minute up to 30,000 lb. and ⅛ in. per minute up to near the maximum load which was maintained for about fifteen seconds.

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[1] Fred E. Foss and R. C. Brumfield, "Some Measurements of the Shape of Brinell Ball Indentation," *Proceedings*, Am. Soc. Testing Mats., Vol. 22, Part II, p. 312 (1922).

The depths of these impressions were measured by mounting an Ames dial on the spindle of a milling machine, placing the rail on the table of the machine and observing the dial readings at the lowest point of the impression, at four points on the edge, and at two points far enough away to escape any effect of the ridge around the edge. The diameters were determined as the average of the diameters parallel to the axis of the rail and at right angles to it, obtained with a steel scale reading to 0.01 in. and a ten power magnifying glass.

TABLE I.—CONVERSION TABLE GIVING BRINELL HARDNESS CORRESPONDING TO DEPTHS AND DIAMETERS OF SPHERICAL IMPRESSIONS MADE WITH A 19-MM. (0.748-IN.) BALL AND 100,000-LB. LOAD.

| Depth, in. | Diameter, in. | Brinell Hardness | Depth, in. | Diameter, in. | Brinell Hardness |
|---|---|---|---|---|---|
| 0.080 | 0.462 | 374 | 0.115 | 0.540 | 260 |
| 0.085 | 0.474 | 352 | 0.120 | 0.549 | 249 |
| 0.090 | 0.487 | 332 | 0.125 | 0.558 | 239 |
| 0.095 | 0.498 | 315 | 0.130 | 0.567 | 230 |
| 0.100 | 0.509 | 299 | 0.135 | 0.575 | 222 |
| 0.105 | 0.520 | 284 | 0.140 | 0.583 | 214 |
| 0.110 | 0.530 | 272 | 0.145 | 0.591 | 206 |

TABLE II.—HARDNESS INDICATED BY DIFFERENT DIMENSIONS OF LARGE IMPRESSIONS AND BY STANDARD BRINELL TESTS.

| Specimen | Brinell Hardnesses as Determined by Various Methods | | | | |
|---|---|---|---|---|---|
| | Depth from Bottom to Top of Ridge, *A* | Depth from Bottom to Level of Original Surface, *B* | Diameter *C* | Depth by Means of Hunt Gage, *D* | Standard Brinell Tests, *E* |
| No. 102 | 286 | 315 | 268 | 272 | 269 |
| No. 103 | 270 | 297 | 251 | 258 | 255 |
| No. 104 | 240 | 261 | 227 | 234 | 241 |
| No. 126 | 262 | 285 | 242 | 249 | 249 |
| No. 130 | 290 | 312 | 270 | 280 | 263 |
| No. 131 | 238 | 275 | 220 | 230 | 238 |
| No. 147 | 275 | 297 | 255 | 265 | 269 |
| No. 166 | 251 | 278 | 240 | 240 | 241 |
| Mean | 264 | 290 | 248 | 253 | 253 |

Average difference individual values of $A$ and $E$ = 4.3 per cent.
Average difference individual values of $C$ and $E$ = 2.7 per cent.
Average difference individual values of $D$ and $E$ = 1.6 per cent.

After this, 0.05 in. of metal was machined from the top of the rail between and at each end of the large impressions, and Brinell tests were made, using the ⅜-in. ball and 3000-kg. load. The diameters of these four impressions were measured with a Brinell microscope by three different observers, each observer measuring a number of different diameters.

*Hardness From Different Dimensions:*

The data in Table II show that the depth of the impression from the level of the original surface to the bottom is useless in calculating the Brinell hardness of the steel as it indicates a hardness much too great. The depth from bottom to top of ridge gives results which also average much too high and the average difference from the standard hardness is 4.3 per cent, which is too great for practical purposes. The average hardness indicated by the diameter of the impressions is close to that indicated by the standard Brinell tests, but the individual results are erratic, the average difference from the standard hardness being 2.7 per cent.

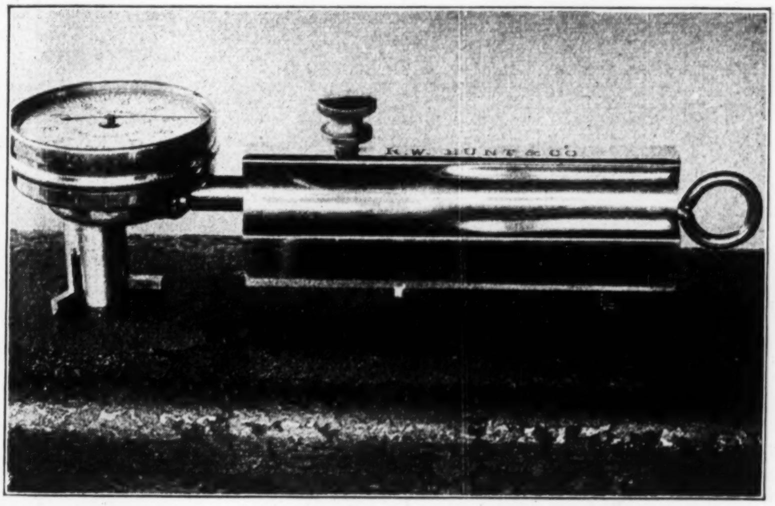

FIG. 2.—Illustration of Hunt Gage for Measuring Large Brinell Impressions.

## BRINELL GAGE

The Hunt gage shown in Fig. 2 was developed to give a reading on the large indentation corresponding to the true Brinell hardness. It consists of a cylinder slotted three ways, in which slides a triangular spider. This spider, by means of a ball-and-socket joint, moves the plunger of a Starrett dial gage reading to 0.001 in. The diameter of this cylinder (0.436 in.) is such that it rests 0.070 in. from the bottom of an impression truly spherical in surface having a spherical diameter of 19 mm. (0.748 in.), so that when the gage is placed on a plane surface, the dial must be set to read seventy. Then when the gage is placed in a Brinell impression, it indicates the distance from the top of the surrounding ridge to a point in the impression where the diameter is 0.436 in. To this, 0.070 in. is automatically added by the setting of the dial. Table I gives corresponding depths, diameters, and hardness. In practice, the gage is adjusted by putting it in an impression in a steel calibration block, the size of the impression having previously been determined, and setting the gage to this figure. This is easier than adjusting it on a plane surface and increases the accuracy of the gage by partially compensating for any wear.

In addition, the gage has the advantage of being extremely simple to use and is not affected by the operator's eyesight or judgment. Many impressions have been measured by different men using different ones of these gages, and almost without exception, have been reported the same within 0.002 in.

TABLE III.—HARDNESS INDICATED BY LARGE IMPRESSIONS MEASURED WITH HUNT GAGE COMPARED WITH HARDNESS FROM STANDARD BRINELL TESTS.

| | Comparative Hardness Values | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Block No. 1 | Block No. 2 | Block No. 3 | Block No. 4 | Block No. 5 | Block No. 6 | Block No. 7 | Block No. 8 | Block No. 9 | Block No. 10 | Block No. 11 | Block No. 12 | Mean |
| Large Impression.. | 260 | 257 | 260 | 252 | 257 | 255 | 253 | 251 | 253 | 255 | 254 | 258 | 255 |
| Standard Tests.... | 260 | 255 | 259 | 259 | 252 | 253 | 249 | 246 | 259 | 246 | 253 | 253 | 254 |

## ACCURACY OF GAGE

In the comparative tests made using the Hunt gage (see Table II), the average hardness indicated by this gage is identical with the average of the standard tests, with an average individual error of 1.6 per cent.

Further evidence of the accuracy of the hardness indicated by large impressions when measured with this gage was obtained when the standard calibration blocks, described in the following paragraph, were made. Twelve of these blocks, measuring 3 by 6 by 1 in. thick were made from rail steel, ground on one side, then one large impression and two standard impressions made there. The standard impressions were measured with a Brinell microscope and the large impression with a Hunt gage and the hardness calculated from these impressions is given in Table III. The average hardness from the large impressions is just one point different from the standard impressions and the average individual error is less than 1.6 per cent. It is evident that the large impressions that were measured with the Hunt gage are accurate indications of the Brinell hardness of the material.

## SOURCES OF ERROR

Impression tests are often made in rail mills by some form of hydraulic testing machine, operated by unskilled labor and with no direct means of checking the load applied. In these cases, the inspectors check the machines by making impressions with them in a rail whose hardness has been previously determined by tests made in a reliable tension testing machine and comparing the size of the impressions.

Experience has shown that there is no perceptible deformation of the balls but questions have occasionally been raised as to the effect of rate of application of load and time of duration of the maximum load. Experiments made in our laboratory show that after the maximum load has been reached, although there is a perceptible creep during the first minute, it does not amount to more than one per cent. As to the effect of rate of application of load, it has been definitely established that this has no appreciable effect even when the rate varies from that given by the slowest speed on a testing machine to a period of application of only a few hundredths of a second, such as is obtained in measuring impact pressures by this method.[1]

## CONCLUSIONS

When the large Brinell impressions made in the head of steel rails with a 19-mm. ball and 100,000-lb. load are measured with the Hunt Brinell gage, the hardness indicated is the same as the hardness indicated by standard size Brinell impressions made on a machined surface. The correct hardness is not indicated by any simple measurement of depth or diameter of the large impression.

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[1] See *Proceedings*, Am. Soc. Testing Mats., Vol. 22, Part II, p. 112 (1922).

# DISCUSSION

**Mr. G. C. Farkell**[1] (*presented in written form by Mr. F. N. Speller*).—This discussion of the paper is based on the use of the impression test during a period of over fifteen years as the important physical test governing the acceptance and rejection of girder rails for street railway purposes. This test has never been completely standardized, various loads, such as 50 gross, 50 metric, and 50 net tons, being used according to the requirements of the various specifications. There have been a limited number of cases during this period in which it has been desirable to translate the results obtained into Brinell hardness numbers which would be closely comparable with the results determined by a laboratory test with the regular Brinell testing machine, using test specimens of which the surface had been prepared for such test.

To do this, a large number of tests were undertaken by our metallurgical department and a curve plotted, the accompanying Fig. 1, using the most commonly used load of 100,000 lb. on a $\frac{3}{4}$-in. ball, the depth of the impression being the depth below the highest original surface of the rail. In commenting on this curve, M. B. Royer of the Metallurgical Department of Lorain Works, states:

> "Careful experimental tests were made on a series of rail specimens of various carbon content by both methods for hardness determination. Approximately 200 individual pieces were tested covering practically the entire range of Class 'A' and Class 'B' rails. The experimental data obtained gave direct evidence that the two tests are not directly comparable in standard Brinell numbers. In all instances it was found that the hardness number calculated from formulas by indentation depth was greater than the corresponding Brinell value obtained by the standard instrument. However, rather constant factors were obtained for both loads, 100,000 and 112,000 lb., whereby the calculated value may be converted directly to the standard hardness number.
>
> ". . . Although the experimental values appear to be somewhat scattered, the curve as drawn approaches the most reasonable and probable value for the hardness number corresponding to a particular depth of indentation.
>
> "It may be stated with reasonable degree of accuracy that the error in hardness numbers from mean values for individual readings will not exceed ±5 per cent of the number. The difference between individual tests may be as much as ±10 per cent in some cases, there being a noticeable tendency for considerable variation between successive individual test pieces. These notice-

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[1] Superintendent, Rolling Mills Dept., Lorain Works, National Tube Co., Lorain, Ohio.

able discrepancies are probably due to the combined effect of various factors affecting the accuracy of the test, which may be:

1. Errors in the mean diameter of impression.
2. Lack of homogeneity of the material.
3. Errors in the measurement of the load.
4. Errors due to the influence of time factor.
5. Errors due to irregularity of rail surface when applying depth gage.

"However, since the values tabulated have been derived experimentally under all the above conditions, by the average of a reasonable amount of readings, the probable maximum variation from the true hardness expressed in standard Brinell values should not exceed ±5 per cent, which would amount to 12 points at the most in hardness."

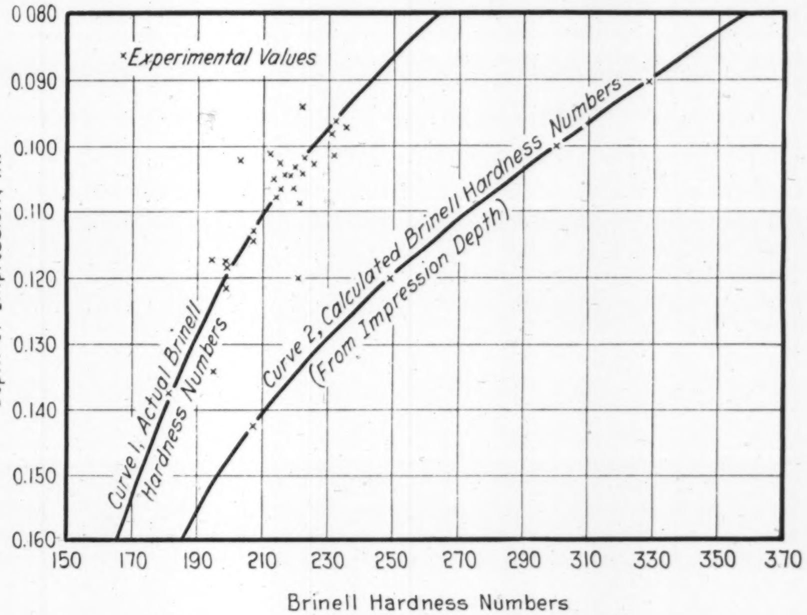

FIG. 1.—Results of Tests Using a 100,000-lb. Load and a $\frac{3}{4}$-in Ball.

We fully agree that when measuring larger impressions, it would be desirable to read directly the result desired. We wish to point out that the purpose of the impression test as adopted and incorporated in the girder rail specifications was not to obtain Brinell hardness numbers but was to obtain the actual depth which a ball loaded with a certain weight will indent the surface of the rail. For this purpose, the gage now used, consisting of a micrometer depth gage recessed for about 1 in. on each side of the impression so as not to interfere with the ridge or bulge which forms around the edge of the impression and using a $\frac{3}{4}$-in. ball to serve for the point of the micrometer screw, is simple and adequate. The test as specified is a practical test which has its value through a comparison of the results of tests made in the same way on different rails. The Hunt gage for measuring large Brinell impressions, described in the paper, should we believe be considered as a means for obtaining the true Brinell hardness numbers and used as a supplement to the standard test prescribed by the specifications.

The results obtained from the limited number of tests as tabulated in the paper indicate close agreement between the hardness number as obtained by this instrument as compared with the standard Brinell test made on the same pieces prepared so as to remove the decarburized surface. In comparing the results obtained as given in Table II of the paper, it is believed that the variations pointed out between *A* and *E* and *C* and *E* are at least partially to be accounted for by the variations in the rough surface of the rail where these tests were made, and that such variations are likely to be found in the case of any type of instrument used, amounting to as much as 5 per cent.

We are unable to satisfy ourselves that the use of a cylinder 0.436 in. in diameter for measuring the depth of the impression below the raised edge is other than an empirical means for bringing about agreement between the observed results and those obtained from tests made on a standard Brinell testing machine, using prepared test specimens.

Without intending to detract from the value of this instrument for converting large impressions directly into corresponding Brinell hardness numbers, we should like to point out that the conditions under which these large impressions are made are so different from a true Brinell test, which conditions are known to give higher loads per unit of impression area, that considerably more work should be done to establish the accuracy of the results not only for a narrow range of hardness but for the complete range to which such a test could be applied.

We are inclined to believe that if a Brinell hardness test is desired, regular test specimens should be prepared and tested under usual conditions.

MR. F. N. SPELLER.[1]—The main point, as you see, is that we recognize the close agreement arrived at by Messrs. Morgan and Mooney between the standard Brinell numbers and the results he has been able to get with his modification. It may be that, by a happy coincidence, the authors have been able to neutralize the well-known basic errors in the impression test, and we hope that this will prove to be the case when the instrument has been applied to a wider variety of product. There are three errors, as our research laboratory points out, that apply to these impression tests. First, the impression form is not spherical but is flattened at the bottom. This is mentioned in Mr. Morgan's paper. Second, there is a fixed relationship between the diameter of the ball and the pressure, which, if departed from, will cause differences in the Brinell numbers obtained; and third, it has been shown that by keeping in mind the necessary relationship between the pressure and the diameter of the ball, the most consistent Brinell numbers are obtained by measuring the depth of impression from the original surface and not from the ridge. The amount of ridge thrown up is dependent upon the pressure, diameter, and in special cases the thickness of the material, so that it is a variable of unknown value.

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[1] Director, Department of Metallurgy and Research, National Tube Co., Pittsburgh, Pa.

MESSRS. H. H. MORGAN[1] AND J. R. MOONEY[2] (*authors' closure by letter*).—When the Hunt gage was designed, there were two objects in view. First, to provide a gage which would be simple to use and reliable under the conditions prevailing at the rail mills and, second, to get an instrument which would give the Brinell hardness of the material directly from a conversion table of depth to hardness. The diameter of the cylinder was fixed at 0.436 in. because this is the diameter corresponding to a depth of 0.070 in. which is a round number, easy to use in setting the dial and also because this diameter is most suitable for making the instrument self centering and gives Brinell numbers direct from the readings.

The Brinell hardness of rails varies from about 200 to 300 and since the size of ball and load employed is used for no other work there seems to be no reason for attempting to determine the accuracy of the instrument over a wider range.

When the impression test was included in the specifications for street railway rails the large ball and load were adopted because it was believed that they would give a more representative indication than a standard size Brinell impression of the hardness of the metal at and just below the surface of the rail which is the part of the rail in which the railway engineers are most interested. The impression depth limits in the specifications are based upon a measurement of the indentation from the bottom to the original level of the rail and any change in the method of measuring the impression should, of course, be accompanied by corresponding changes in these specified limits.

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[1] Manager, Rail and Fastenings Dept., Robert W. Hunt Co., Chicago, Ill.

[2] Assistant, Rail and Fastenings Dept., Robert W. Hunt Co., Chicago, Ill.

The Brinell number is the standard way of expressing the hardness of metals and the large impressions when measured with the Hunt gage give reliable and accurate indications of the Brinell hardness of the rail head and surface. The street railway engineers are much interested in the wear of their rails and consequently in the hardness of the rail at the surface of the head, and since it is now possible to get this information directly from the large impressions being made on rails we believe that it should be done.

# HIGH-TEMPERATURE CHARACTERISTICS OF METALS REVEALED BY BENDING

By Howard Scott[1]

## Synopsis

The curvature produced in an initially flat strip by heating it while bent to a known curvature is easily measured after release at room temperature. The acquired curvature is found from analytical considerations to be a measure of the plastic deformation which occurred at the test temperature. As the curvature change can be easily measured with high sensitivity and the testing equipment is simple, it offers an attractive means for investigating the plastic behavior of metals at elevated temperatures.

When this test method was applied to phosphor bronze, a critical strain which corresponds to the strain at the proportional limit of the same material when tested in tension at moderate temperatures was easily recognizable and readily evaluated. At lower values of the applied strain, the plastic deformation was found to be proportional to the applied strain which corresponds to the load in the case of a tension test. This effect is, of course, small in magnitude at room temperature but increases exponentially with temperature.

These observations suggested the concept of a critical temperature below which the plastic deformation is less than some small fraction of the total strain. Several metals were compared on the basis of that critical temperature which appears to be a fundamentally important factor entering into the selection of metals for high-temperature service.

While keeping the temperature and applied strain constant, the variation in plastic deformation with time was studied. After the first few hours and up to 100 hours at least, the plastic deformation increased logarithmically with time. The initial flow increased very much more rapidly with temperature than did the rate of flow after steady flow was established. These observations are consistent with tensile creep test results and suggest that the bend test can provide results which are directly comparable with those of the creep test under equivalent test conditions.

## Introduction

In order to broaden our knowledge of the elastic properties of metals at high temperatures, it is necessary to raise the upper temperature limit of present testing practice. With the conventional tension test that limit is restricted to temperatures at which the best available alloys retain their strength to a substantial degree. This limitation is imposed by the need for making strain observations at the test temperature and for using machineable alloys in extensometer parts. Consequently, any method of evaluating strength characteristics which does not require strain observations at the test temperature opens up the possibility of extending greatly the range of testing temperature.

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[1] Section Engineer, Research Laboratories, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

Loading of strip specimens by bending eliminates the need for strain observations at the test temperature. If an initially flat strip is heated as constrained to a constant curvature, the applied strain is easily calculable from the thickness of the specimen and its curvature, which dimensions are readily measured at room temperature. The applied strain remains unchanged with temperature so long as there is no distortion in the restraining fixture. That fixture is loaded chiefly by its own weight alone so distortion presents no problem. Also, it may be made of a ceramic material if exceptionally high-test temperatures are required.

A significant dependent variable is furnished by the residual curvature of the strip as released at room temperature after heating in constraint. This curvature may also be easily measured with high accuracy at room temperature. It evidently results from plastic deformation at the test temperature so there are available in the applied strain and residual curvature of bent strips the equivalents of important components of the tensile stress-strain curve.

If the bending variables can be explicitly correlated with the equivalent variables of the stress-strain curve, the prospects for obtaining useful information from the simple observations mentioned are excellent. One correlation is immediately evident. No appreciable residual curvature will be produced at normal temperatures until the applied strain exceeds that at the proportional limit of the metal. Hence, by varying the applied strain at constant temperature, the proportional limit or an equivalent property can be evaluated in strain units.

The values of strain at the proportional limit obtained by bending should coincide with those obtained by the tension test if the basic principles of the bend test are sound. Intercomparison of the test methods, therefore, permits a check on our interpretations of bend test results. Strain units can be easily converted into stress units if the modulus of elasticity of the metal tested is known. Even if its value in a particular alloy at elevated temperature is not known, it can be estimated with some assurance from knowledge of the moduli of the component metals because this property appears to follow the rule of mixtures in most alloys. In the light of these considerations and the possibility of developing a simple, flexible and inexpensive high-temperature test, the matter has seemed worthy of detailed study.

## PRIOR WORK

The idea of exposing a flat strip constrained at a known curvature to a certain temperature for evidence of its elastic properties at that temperature appears to have originated with Moore and Beckinsale.[1] Their object was to determine the lowest temperature at which residual stresses in cold-drawn brass can be completely eliminated, a very appropriate application of the idea. They worked at stresses much above the yield point, but apparently did not recognize the fact that the stress calculated from the residual curvature with the usual bending formula is not the true stress at the test temperature.

Masing[2] used the same test method and demonstrated an approximate relation between the apparent stress at loads beyond the yield point and the actual stress at the test temperature, assuming a simple ideal stress-strain curve. Moore and Beckinsale's values of the maximum stresses as bent are high, while Masing's are low. Neither of these investigators used the test with the view of evaluating the proportional limit or creep limit of metals.

The author had occasion to investigate bending at elevated temperatures with special regard to residual stresses. In the course of that investigation, Mr. P. H. Brace suggested the possibility of applying bending observations to the quantitative evaluation of the strength of metal at high temperatures with particular reference to certain high-strength, high-temperature alloys which were being investigated. Analytical considerations sustained the practicability of the test method and it was accordingly submitted to experimental trial.

## BASIC PRINCIPLES

When a flat strip of rectangular cross-section is bent to a constant curvature, the stress developed is a maximum at the curved faces. It is given for elastic loading by:

$$p_m = E\,c\,V \qquad (1)$$

where $E$ is the modulus of elasticity of the metal; $c$ the half thickness of the strip; and $V$ the curvature of the form, that is, the reciprocal of the radius of curvature of the cylindrical surface to which the strip is constrained. The stress is tensile on the convex side of the neutral axis and compressive on the concave side. It varies linearly between those faces, passing through zero at the neutral axis.

If now the curvature of constraint be increased until the proportional limit is exceeded in the outer fibers a limited amount of plastic flow will occur and the stress distribution will be altered in a manner determined by the stress-strain curve of the metal, as shown in Fig. 1 (*a*). The maximum stress is then less than would be obtained from Eq. 1. Its magnitude may be found from the stress-strain curve used, curve *A*, Fig. 2, by taking the stress corresponding to the maximum strain applied. That strain from Eq. 1 is *Vc* and is identified by the term "applied strain." Now when released the strip cannot return to its original stress-free condition because the stress loss on release is elastic, that is, it follows the modulus curve $op_1$. Consequently, residual stresses remain as shown in Fig. 1 (*b*). The prin-

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[1] H. Moore and S. Beckinsale, "The Prevention of Season Cracking in Brass by the Removal of Internal Stress," *Transactions*, Faraday Soc., Vol. 17, p. 162 (1921).

[2] G. Masing, "Das Aufreissen von Messing durch innere Spannungen," *Zeitschrift für Metallkuend*, Vol. 16, p. 257 (1924).

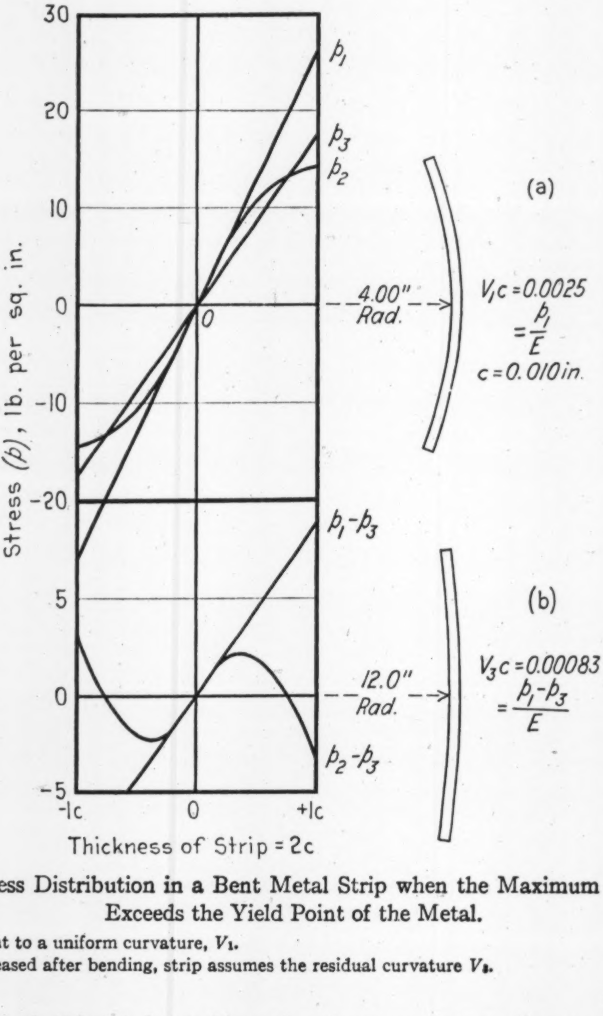

FIG. 1.—Stress Distribution in a Bent Metal Strip when the Maximum Stress, $p_1$, Exceeds the Yield Point of the Metal.

(*a*) As bent to a uniform curvature, $V_1$.

(*b*) As released after bending, strip assumes the residual curvature $V_3$.

ciples of such residual stress generation have already been fully described by Timoshenko[1] and others.

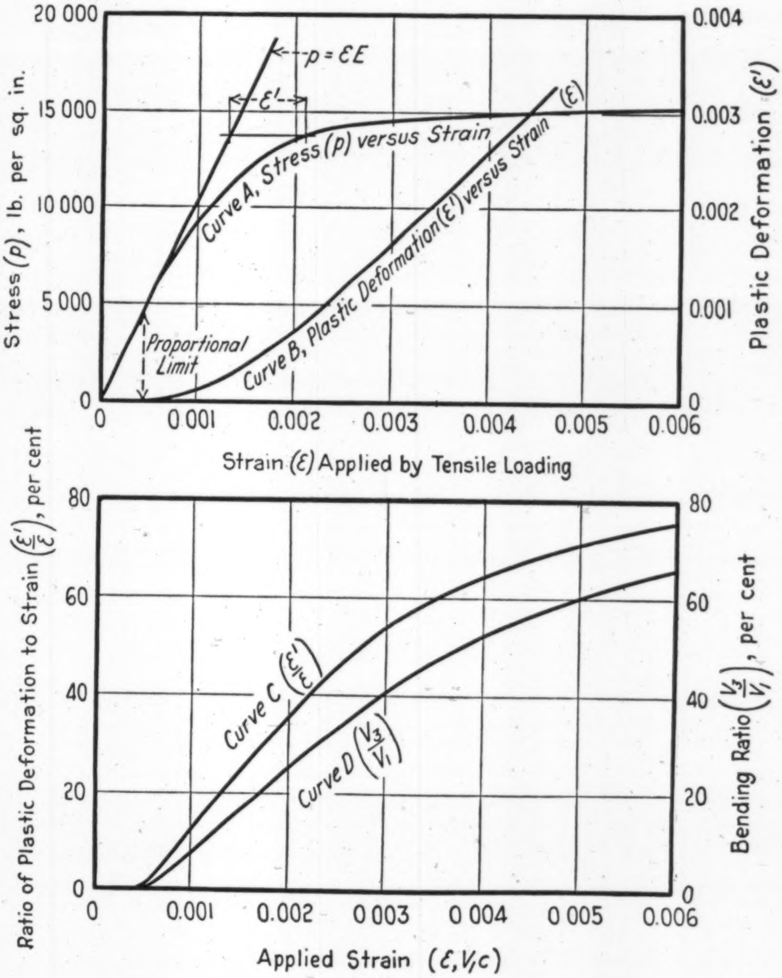

FIG. 2.—Relation of Bending Ratio to Stress-Strain Curve of a Metal.

Curve *A*, observed stress-strain curve.

Curve *B*, plastic deformation, that is, difference in strain units between stress-strain curve and modulus curve extended.

Curve *C*, plastic deformation in per cent of applied strain.

Curve *D*, calculated bending ratio, that is, the curvature change in per cent of the curvature as bent.

The problem of estimating the residual curvature is the same as that of estimating the residual stresses. It requires the evaluation of $p_3$ when this term has such a value that the moments of the areas included between $op_3$ and $op_2$ are zero. Then $p_3$ will go to zero when the strip is released and the residual curvature will be determined by $(p_1 - p_3)$ where $p_1 = EcV_1$ and $V_1$ is the curvature of constraint. Hence, the residual curvature, $V_3$, is given by:

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[1] S. Timoshenko, "Strength of Materials," Chapter 7, Part I, D. Van Nostrand Co., New York City, 1930.

$$V_3 = \frac{p_1 - p_3}{Ec} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Now $p_1$, $p_3$ and $E$ can be evaluated in any case from knowledge of the stress-strain relation, $V_3$ can be determined by curvature measurement of a bent strip, and hence the validity of Eq. 2 can be tested experimentally.[1]

The preceding findings may be used at once to illustrate the relation between the residual curvature produced by bending and the plastic deformation observed on testing in tension. For that purpose the residual curvatures expected from bending a material having the stress-strain curve, curve $A$, of Fig. 2 was calculated. The observed plastic component of the tensile strain, $\mathcal{E}'$, is the difference in strain units at a given load between the observed curve and its modulus curve extended. Plotting that characteristic against strain yielded curve $B$, Fig. 2. It is just as difficult to pick a proportional limit from that curve as from the original stress-strain curve.

If now the plastic deformation be plotted in per cent of the applied strain as in curve $C$, Fig. 2, the proportional limit stands out very sharply. This method of plotting is particularly adapted to the presentation of residual curvature observations, curve $D$, Fig. 2, because only the equivalent of the plastic component of tensile strain is measured and that with high precision. Since the ratio of residual curvature to the curvature of constraint is used extensively in the following text, it is identified by the term "bending ratio." It is expressed in terms of stresses by:

$$\frac{V_3}{V_1} = \frac{p_1 - p}{p_1} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

where $p_1$ and $p_3$ are evaluated in accordance with the preceding discussion.

Comparing the bending ratio curve, curve $D$, Fig. 2, with that of tensile plastic deformation, curve $C$, Fig. 2, it is immediately evident that they have the same shape, but that the curve for the bending ratio falls below the other except at the proportional limit and below where they merge. Consequently, the observed values of the bending ratio will always be lower than the corresponding values of the ratio of plastic to total strain obtained in a tension test above the proportional limit of a metal having the type of stress-strain relation under discussion.

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[1] A graphical method of evaluating $p_3$ is given in the appendix, see p. 156.

One is not justified in calling the applied strain at which the bending-ratio curve first swings upward the proportional limit of the metal in strain units because the bending ratio yields no evidence of the shape of the true modulus curve. In fact, Sayre[1] has observed a definite change in slope of the modulus curve with loads so low as to produce no detectible permanent deformation, an observation which makes the existence of a "proportional limit" a fiction in rigorous usage. Consequently, to avoid any implications as to the shape of the modulus curve, it is desirable to identify the applied strain at which the bending ratio curve first swings upward by a different and distinctive term. For that purpose the term "relaxation strain" is used.

The practical significance of the comparison between the bending ratio and tensile plastic deformation is that it shows the former property to be a measure of the latter. Of the total deformation in a tension test, a certain portion, $\mathcal{E}'$, of curve $A$, Fig. 2, is plastic. The residual curvature of bending is a result of the corresponding plastic deformation and when multiplied by the half thickness of the strip the residual curvature yields an apparent value of the maximum strain lost by plastic deformation. The apparent value of that plastic strain is, from curves $C$ and $D$, Fig. 2, somewhat lower than the plastic deformation under tensile loading to the same applied strain. In an important limiting case, however, it is numerically equal to the tensile plastic deformation.

The limiting case mentioned is realized when the observed stress-strain relation in tension is given by a straight line of lower slope than the ideal modulus curve, namely, by $op_3$ of Fig. 1. Then the ideal modulus curve $op_1$ is approximated by the stress-strain relation on unloading. It is not necessary to assume that both curves are straight lines, but only that the separation is proportional to the strain applied. In that event, the apparent plastic strain of bending is numerically equal to the tensile plastic deformation with the same applied strain. If then a constant value of the bending ratio be observed over a range of applied strains below the relaxation strain, it may be concluded that the apparent plastic strain of bending and the equivalent tensile plastic deformation have identical values.

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[1] M. F. Sayre, "Elastic Behavior of Spring Materials," *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part II, p. 546 (1930).

That situation, or a close approach thereto, is frequently found experimentally, with the result that the interpretation of bend test results is much simpler than anticipated.

In these considerations it is assumed that the stress-strain curve in compression is identical with that in tension. Substantial departure from this relation is rare except in metals which have received unidirectional plastic deformation. In them the Bauschinger effect, that is, the lowering of the proportional limit in compression by deformation in tension and *vice versa,* introduces uncertainty and obscures the inherent characteristics of the metal. Consequently, it is necessary to assume that all specimens of unknown preparation have undergone some plastic deformation and treat them accordingly.

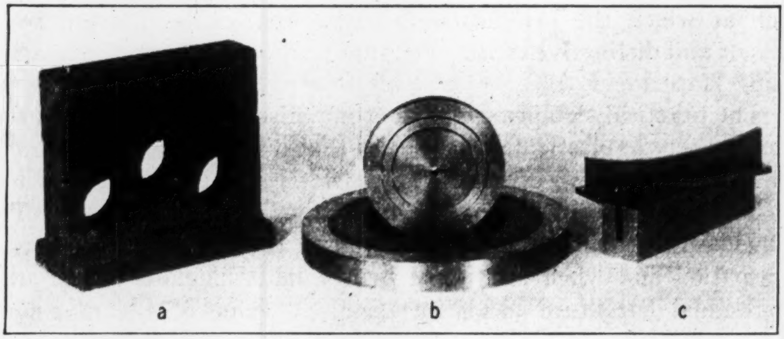

FIG. 3.—Three Types of Test Forms Used to Constrain Strip Specimens While Heating for Test.

Fortunately, the effects of inadvertent plastic deformation can be removed even in cold-worked metals without appreciable modification of their inherent elastic strength by mild heat treatment. We have, therefore, in the bend test a means for studying the effects of cold working on strength of metals at high temperatures, a project hardly feasible with the tension test because of the large test samples required.

## EXPERIMENTAL METHOD

For the application of the high-temperature bend test, forms are required to constrain the strip specimens to a pre-determined curvature while at the test temperature. Several types of forms are shown in Fig. 3 of which *a* is preferable for large curvatures and *b* for small. The best type for general service is probably *c.* It is sufficiently massive to be rigid and still light enough to permit rapid heating of the specimens and precise realization of a desired temperature cycle.

Also it minimizes the chances of accidental bending on placing and removing the specimens. If made from heat-resisting alloys and properly supported in the furnace, the forms may be used at temperatures up to 900° C. without serious distortion.

The essential requirements in the preparation of test specimens are the removal of the effects of previous incidental deformations and the securing of flat specimens. Both objectives are accomplished most effectively by two operations. First, the strip stock is submitted to reversed bending of diminishing amplitudes by drawing between rolls or pins so adjusted that it comes out flat. Second, the test specimens cut from the flattened stock are heated as constrained flat to a temperature above the test temperature. That temperature need not be as high as the recrystallization temperature of the test metal even when severely cold worked, so cold-worked metals can be pre-treated without material permanent loss in hardness.

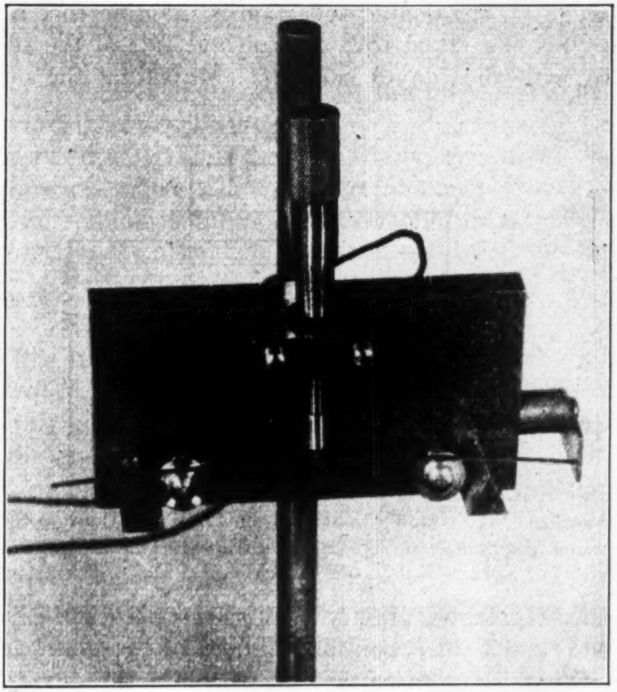

**FIG. 4.—Device Used for Measuring Curvature of Specimens.**

The curvature of the test specimen is measured both before and after heating in the test form. A device for measuring curvature is shown in Fig. 4. The strip specimen is supported on two parallel knife edges and its vertical displacement from the plane of the knife edges is measured midway between them with a ball-pointed micrometer head. From that displacement, commonly called deflection, the curvature, $V$, of the strip is calculated by means of the relation:

$$V = \frac{4d}{L^2 + d^2} \quad \text{(4)}$$

where $d$ is the double deflection of the strip and $L$ the distance between the knife edges. The double deflection is the difference between the micrometer reading taken with the concave side of the strip up and that taken with the convex side up. Measuring the deflection in that manner eliminates gravity effects, the need for establishing a datum point, and a correction for thickness of the specimen.

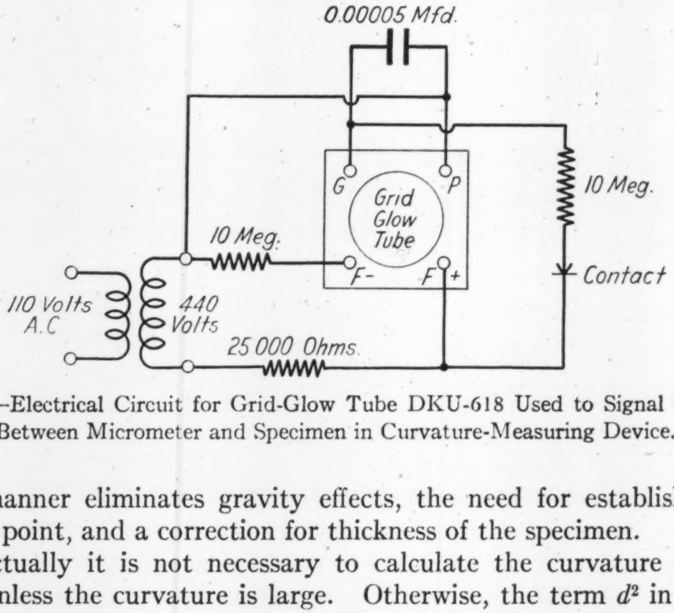

FIG. 5.—Electrical Circuit for Grid-Glow Tube DKU-618 Used to Signal Contact Between Micrometer and Specimen in Curvature-Measuring Device.

Actually it is not necessary to calculate the curvature of the strip unless the curvature is large. Otherwise, the term $d^2$ in Eq. 4 is so small that it can be neglected. Then the bending ratio is given by:

$$\frac{V_3}{V_1} = \frac{d_3}{d_1} \quad \text{(5)}$$

where $d_3$ is the double deflection of the released strip and $d_1$ its double deflection as constrained. If, however, the strip has appreciable initial curvature, it is necessary to introduce that variable into the calculation of the bending ratio by using the following equation for the bending ratio:

$$\frac{V_3 - V_0}{V_1 - V_0} = \frac{d_3 - d_0}{d_1 - d_0} \quad \text{(6)}$$

where $V_0$ is the initial curvature and $d_0$ the corresponding double deflection. Now one must distinguish between positive and negative curvatures which is best done by considering strip curvatures taken in the same direction as that of the form as positive and in the opposite direction as negative.

Deflection can be measured as illustrated with great precision by using a grid-glow tube to indicate the establishment of contact between the micrometer head and the test specimen. Using the circuit given in Fig. 5, a movement in the micrometer head of 0.0001 in. is sufficient to make or break the contact. No observable pressure is required to establish a contact which will be revealed by a red glow in the tube. This high sensitivity of the grid-glow tube is not so important in high-temperature bending testing because the residual deflections are usually high, but its ability to operate through the high electrical resistance of a thin oxide film is then valuable. That charactersitic eliminates the need for cleaning or pickling specimens after heating and the attendant danger of accidental bending.

## TEST RESULTS

With the high-temperature bend test, as with the tension test, there are three important independent variables, namely, load, temperature and time. Load in the case of the bend test of course refers to the applied strain. The effects of each of these variables may be shown clearly by discussing one at a time, while the other two remain constant.

### *Load Variable:*

The most complete bend test data available were obtained on phosphor bronze. This material was cold drawn and then rolled cold into ribbon with a net reduction in area after the last anneal of about 80 per cent. So prepared, recrystallization starts on tempering at about 325° C. Accordingly, the test specimens were prepared for test by heating constrained flat at a somewhat lower temperature, 300 to 310° C. With this preparation, the relaxation strain is probably least distinct so the test results are representative of the most difficult conditions for evaluation of that property.

Observations of the bending ratio taken at atmospheric temperatures are plotted against applied strain in Fig. 6 to afford a comparison with tension test results. The load range extended only slightly beyond the relaxation strain and the plastic deformation is small under these conditions so a very open scale was used in plotting the results. The tensile proportional limit of duplicate specimens was determined by Mr. P. L. Irwin, formerly of these laboratories, and the values obtained are given in strain units in the figure. As was anticipated, those values agree well with the upswing of the corresponding bending-ratio curves marking the relaxation strain. The correspondence is best as it should be in the specimen which showed the most clear-cut proportional limit, namely, the one tempered at 400° C.

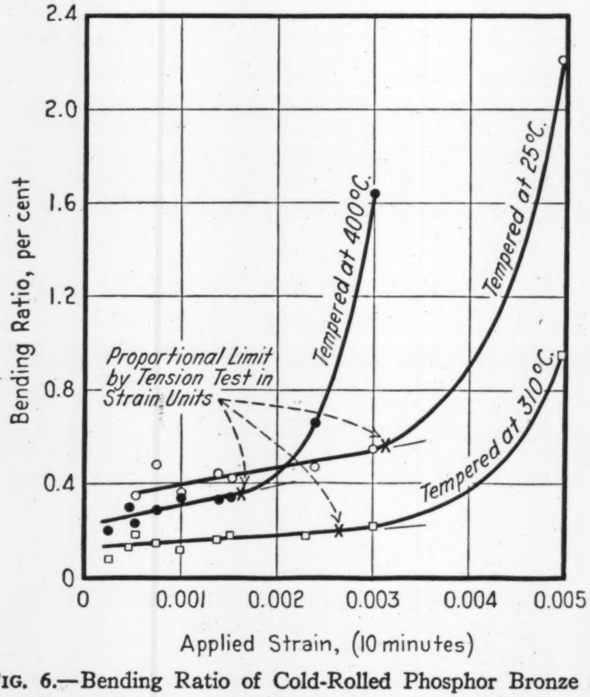

FIG. 6.—Bending Ratio of Cold-Rolled Phosphor Bronze at Room Temperature Plotted Against Applied Strain.

A prominent feature of the bending-ratio curves is the plastic deformation observed at loads below the relaxation strain. This phenomenon, usually called permanent set, is familiar to spring designers,[1] but escapes detection in the tension test as ordinarily made. The tension test fails to reveal it because its magnitude is very small at room temperature, because the time under load is small, and particularly because the plastic deformation produces no appreciable departure from the nominal linearity of the stress-strain curve.

In the discussion of basic principles, it was shown that a constant value of the bending ratio over a range of loads can occur only when the plastic deformation is proportional to the load. The observed bending ratio below the relaxation strain in Fig. 6 substantially fulfills that condition, so the associated plastic deformation will not be revealed by appreciable curvature of the stress-strain curve. That plastic deformation will nevertheless influence the apparent modulus of elasticity, the observed value of which will be lower than the true value by the amount of that plastic deformation when the true value is defined as that obtained with zero plastic deformation. The difference between the true and observed values, however, is small at atmospheric temperatures, certainly less than 1 per cent, but may attain an important magnitude at elevated temperatures.

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[1] B. W. St. Clair, "Springs for Electrical Measuring Instruments," *Mechanical Engineering*, Vol. 47, p. 1057, (1925).

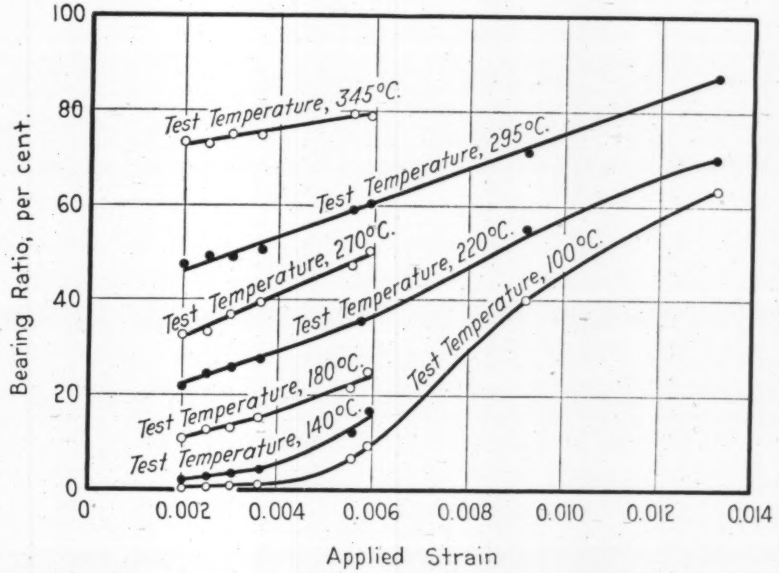

**FIG. 7.—Bending Ratio Curves of Cold-Rolled and Tempered Phosphor Bronze Tested at Elevated Temperatures. Tempered at 310° C.**

**Time at test temperature, 15 minutes.**

Bending-ratio curves plotted from data taken at elevated test temperatures and high loads following the technique described in the preceding section are given in Fig. 7. The curves obtained at the higher test temperatures are much the same as those at room temperature, but there are significant changes in details. For example, the degree of plastic deformation below the relaxation strain increases very rapidly with temperature and the bending-ratio curve revealing it acquires considerable slope. One may expect, therefore, greater curvature of a stress-strain curve taken under comparable conditions at loads below the proportional limit when testing at high temperatures than when testing at low temperatures, and greater departure of the observed modulus from the true modulus, a matter discussed later.

Considering Fig. 7 from the viewpoint of relaxation strains, it appears that for cold-worked phosphor bronze this characteristic vanishes from within the range of applied strains used when the temperature exceeds 200° C. Thus, at 295° C. the bending ratio varies linearly with load between the extreme limits of applied strain, namely, 0.002 and 0.013. These observations suggest that possibly the relaxation strain drops to values below the range of applied strains used. That possibility was investigated by taking observations on the same material at low values of applied strain with results presented in Fig. 8. Actually, the values of permanent bending and the slope of the bending-ratio curves increase with temperature so that the inflection in the curves marking the relaxation strain becomes indistinct and difficult to recognize at 200° C. and higher. Moreover, the considerable slope of the permanent bending curves at those tempera-

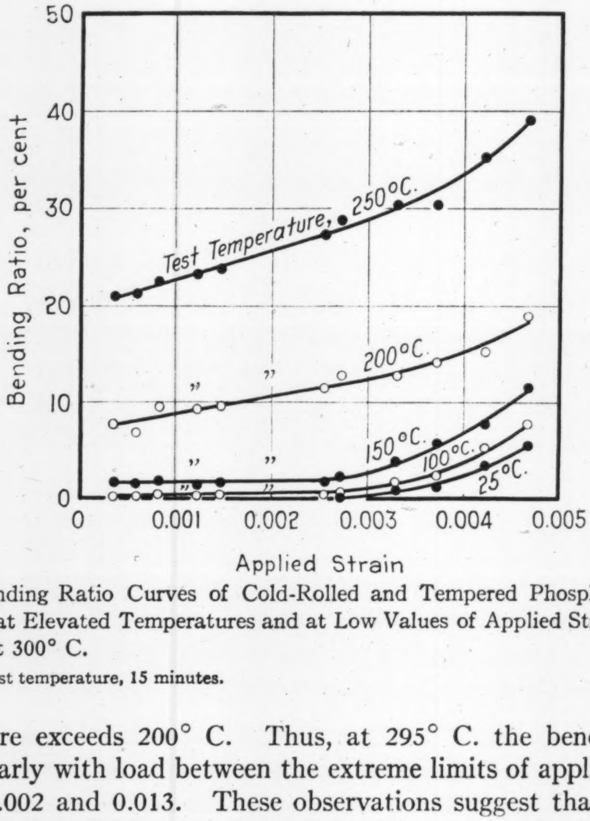

FIG. 8.—Bending Ratio Curves of Cold-Rolled and Tempered Phosphor Bronze Tested at Elevated Temperatures and at Low Values of Applied Strain. Tempered at 300° C.

Time at test temperature, 15 minutes.

tures show that the stress-strain curve will have appreciable curvature so that the value of the strain at the proportional limit will probably be lower than the relaxation strain. These considerations suggest that the proportional limit as determined by tension tests at temperatures where the permanent bending curve has considerable slope does not have the same significance as when determined at room temperature.

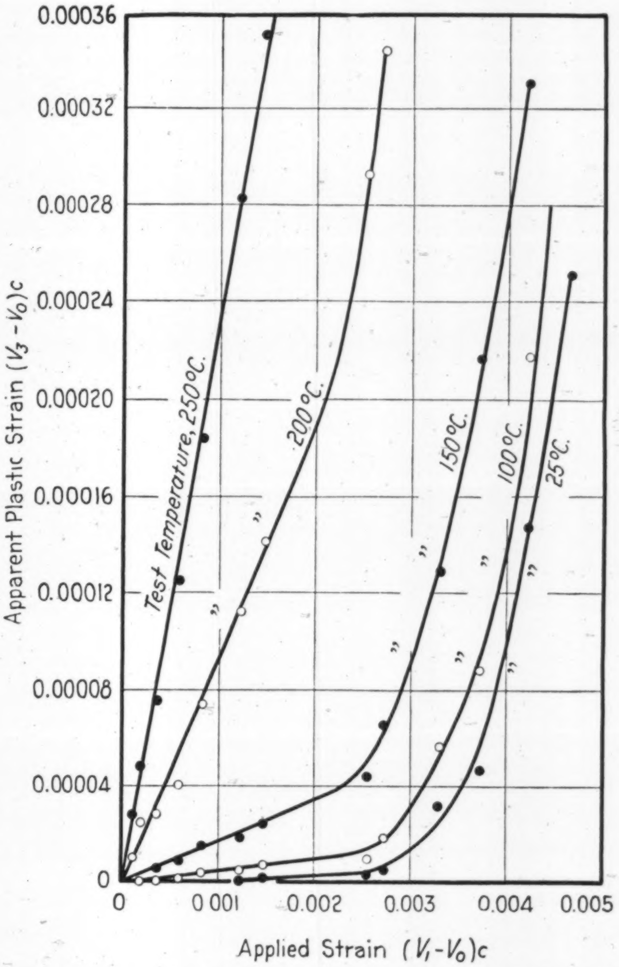

FIG. 9.—Showing the Relation of Apparent Plastic Strain to Applied Strain for Low Values of Applied Strain.

Time at test temperature, 15 minutes.

The foregoing observations introduce the question: "Is there any load below which plastic deformation does not occur when temperatures are high enough to cause the plastic deformation at the relaxation strain to be appreciable?" That question may be answered by plotting the apparent plastic strain, that is, the residual curvature multiplied by the half thickness of the specimen, against applied strain as in Fig. 9 to provide a dimensionless criterion of plastic deformation. It should be noted that the ordinate units are one-twenty-fifth of those of the abscissa.

Again in Fig. 9 the curves for low loads are linear or nearly so, as they must be when the bending-ratio curve has no slope. There is here evidence of a limit to which the apparent value of the proportional limit, or in this case the relaxation strain, can be reduced by increasing the sensitivity of the test method. At 100 and 150° C. the values of the apparent plastic strain are much greater than the observational error, so if the curvature of those curves changed continuously with applied strain, that situation would be quite evident. The considerable range of linearity followed by a pronounced increase in slope, however, suggests strongly that there is a change in the law of plastic deformation associated with that inflection in the curves. Thus, at moderately high temperatures, conditions are relatively more favorable for determining whether or not the relaxation strain is a fundamental constant of a metal.

TABLE I.—COMPOSITION AND PREPARATION OF ALLOYS USED IN BEND TEST.

| Composition | Thickness, in. | Coal-Reduction in Area, per cent | Remarks | Figure |
|---|---|---|---|---|
| 5 per cent Tin Bronze | 0 008 | 87 | 75 per cent reduction in area as wire. | 6 |
| | 0.016 | 93 | | 6 |
| 2 per cent Tin Bronze | 0.012 | 88 | 75 per cent reduction in area as wire. | 7 |
| 2 per cent Tin Bronze | 0.015 | 84 | 69 per cent reduction in area as wire. | 8, 9, 14, 15 |
| | 0.020 | 80 | | 14, 15 |
| | 0.028 | 75 | | 14, 15 |
| 0.5 per cent Carbon Steel | 0.017 | Annealed at 950 to 1000° C. | ........................ | 10 |
| 80–20 Nichrome | 0.017 | Annealed at 950 to 1000° C. | ........................ | 10 |
| Nickel-Cobalt-Titanium Alloy | 0.017 | Age hardened | K-61 | 10 |
| | | | K-32 | 10 |

If now the linear portions of the curves in Fig. 9 be extended to zero apparent plastic strain, the curves for 100 and 150° C. will cross the axis of applied strain at finite positive values. This fact may be taken to mean that at these and lower temperatures phosphor bronze has a true elastic limit in the sense that it may be loaded without the occurrence of irrecoverable plastic deformation. If so, the stresses are very low, not over 2500 lb. per sq. in. at 100° C. or 900 lb. per sq. in. at 150° C. At 200° C. and above, plastic deformation will probably occur at any finite load.

*Temperature Variable:*

The most significant fact disclosed in the preceding section is that at elevated temperatures the plastic deformation at loads below the relaxation strain is a considerable proportion of the total strain.

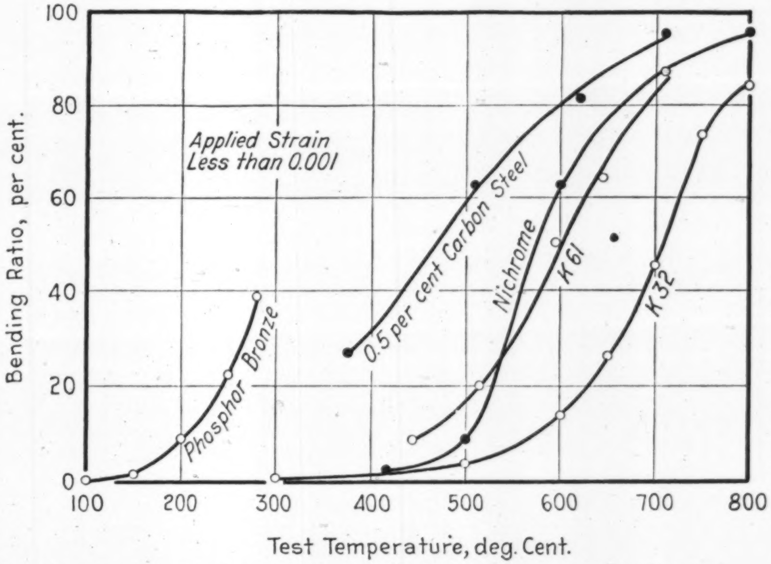

FIG. 10.—Bending Ratio of Several Alloys Plotted Against Test Temperature.

The values of applied strain used were constant for each material and were well below the values at the proportional limit cold. Time at test temperature, 15 minutes.

Since the bending ratio is a measure of the plastic deformation, it may be plotted against temperature to advantage. Values for a number of alloys described in Table I are so plotted in Fig. 10. The applied strain was of course held constant at the values noted, which were well below the relaxation strain at normal temperature, so the relative positions of the curves on the temperature scale are not determined to any great extent by that factor.

The curves of Fig. 10 are evidently characteristic of the metals concerned when the applied strain and time at the test temperature are fixed. Such curves provide a much needed means for classifying metals as regards their upper temperature limit of mechanical utility. Classification can be further simplified by taking a single temperature at which the bending ratio reaches some arbitrary value. Since the curves all have much the same shape, that temperature is representative of the relative positions of the curve on the temperature scale.

The literature supplies precedent for the foregoing concept of a critical temperature revealed by the variation in mechanical properties of metals with temperature. Ingall[1] finds a well-defined critical temperature in the variation of tensile strength with temperature. Lees, Andrews and Shave[2] call attention to a break in their curves of elastic modulus plotted against temperature which marks the intrusion of plastic deformation. The relation between that break in the modulus curve and the permanent bending of phosphor bronze is shown by plotting to a common temperature scale the data of Andrews[3] and of the author as in Fig. 11. It may be seen that the break in the modulus curve occurs at a temperature close to the probable temperature of maximum slope of the permanent bending curve which could not be determined without producing recrystallization in the test specimens.

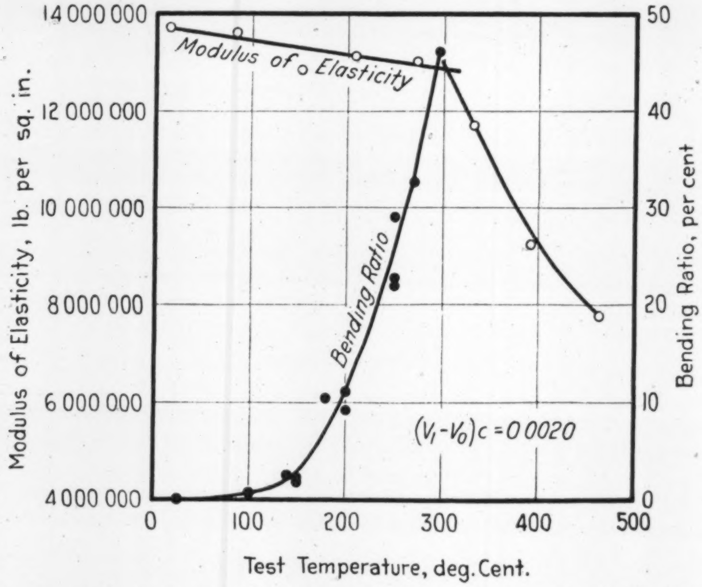

FIG. 11.—Variation in the Elastic Modulus of Phosphor Bronze with Temperature Compared with the Variation of Bending Ratio.

The modulus values were taken from Andrews' paper.

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[1] D. H. Ingall, "Experiments with Some Copper Wire: Cohesion a Function of Both Temperature and Cold Work," *Journal*, Inst. Metals, Vol. 30, p. 171 (1923).

[2] C. H. Lees, J. P. Andrews and L. S. Shave, "The Variation of Young's Modulus at High Temperatures," *Proceedings*, Physical Soc. London, Vol. 36, p. 405 (1924).

[3] J. P. Andrews, "The Variation of Young's Modulus at High Temperatures," *Proceedings*, Physical Soc. London, Vol. 37, p. 169 (1924).

It is desirable, therefore, to select some point on the curve more nearly representative of the upper temperature limit of its mechanical stability under load than is the temperature of maximun slope. Referring to Fig. 8, it may be seen that cold-worked phosphor-bronze retains a fairly high relaxation strain at 200° C., at which temperature the bending ratio below the relaxation strain reaches a value of 10 per cent. It is suggested, therefore, that the temperature at which the bending ratio reaches a value of 10 per cent under specified conditions of applied strain and time at test temperature offers an appropriate, although an admittedly arbitrary single-valued criterion of the temperature limit of mechanical utility which will be designated by "relaxation temperature." It should be recognized, however, that the property so designated conveys no implication as to the strength of the metal at that temperature.

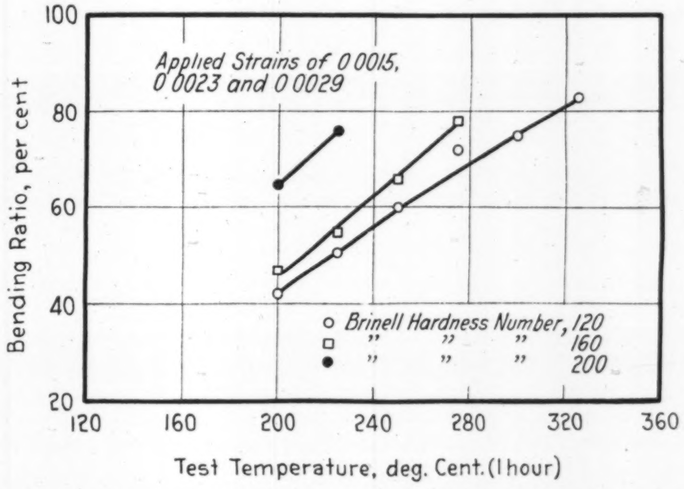

FIG. 12.—Bending Ratio of 70–30 Brass Cold-Rolled to Three Different Hardness Values Plotted Against Temperature from the Data of Moore and Beckinsale.

The modulus curve of Fig. 11, having been determined under conditions which minimize plastic deformation, offers pertinent information on the change in modulus with temperature. Andrews places the onset of plastic deformation at 300° C. from the break of the curve. The bending ratio at that temperature is 50 per cent, while the drop in the modulus from its value at room temperature is only 6 per cent. That drop might be attributed entirely to increased plastic deformation with increasing temperature. In that event the slope of the curve should be lower at low temperatures, at least below

150° C., for the plastic deformation as shown by the bend test is then comparatively small. It is probable, therefore, that Andrews' modulus curve gives a close approximation to the variation in the true modulus of phosphor bronze with temperature. Certainly the true

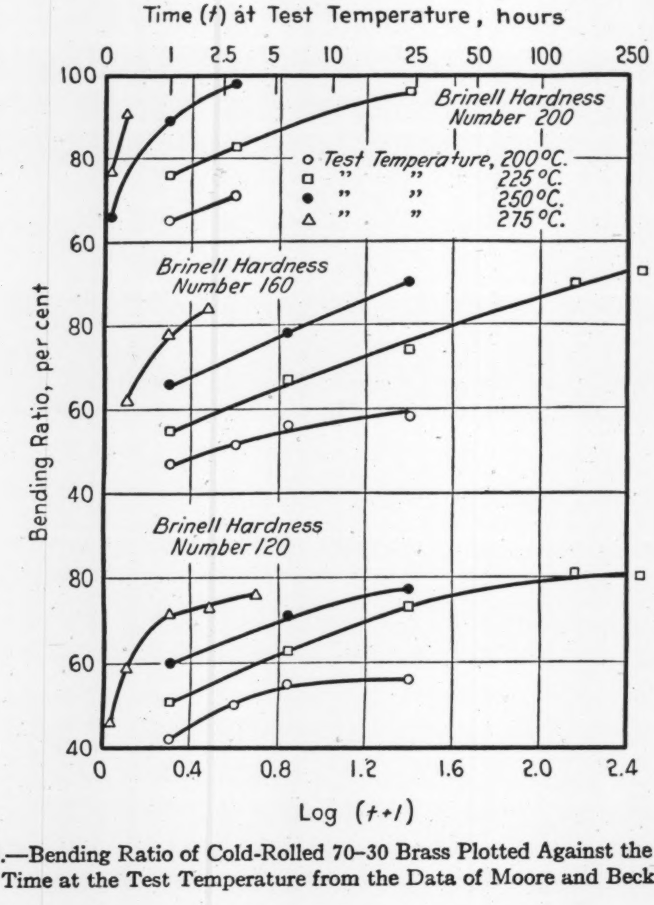

**Fig. 13.—Bending Ratio of Cold-Rolled 70–30 Brass Plotted Against the Logarithm of Time at the Test Temperature from the Data of Moore and Beckinsale.**

modulus value does not drop more than 6 per cent at 300° C. so there is no difficulty in estimating the stress up to and at the relaxation strain with reasonable accuracy in metals whose moduli are known for normal temperatures.

The data of Moore and Beckinsale throw light on the effects o an important metallurgical variable, namely, the degree of cold working. Their observations on the bending ratio of 70–30 brass cold rolled to three different Brinell hardness values are plotted against temperature in Fig. 12. Each observation is the mean of three values of applied strain, each of which was below the relaxation strain of the softest strip when cold. At the same test temperature the value of the bending ratio is greater the harder the brass. Evidently then, cold working is favorable to plastic deformation at high temperatures—which is contrary to our experience at normal temperatures. It corresponds, however, with the observation of Kanter and Spring[1] that

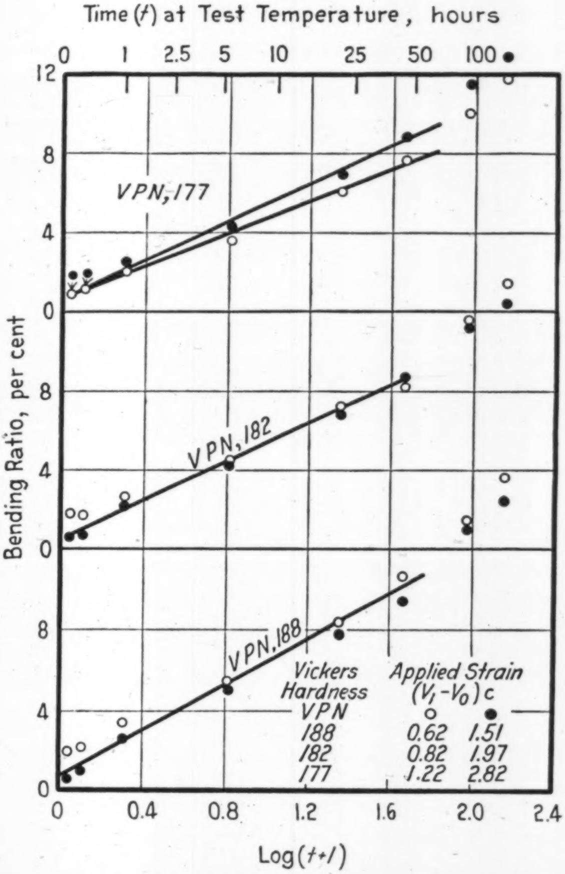

FIG. 14.—Bending Ratio of Cold-Rolled Phosphor Bronze Plotted Against the Logarithm of Time at the Test Temperature of 150° C.

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[1] J. J. Kanter and L. W. Spring, "'Long-Time' or 'Flow' Tests of Carbon Steels at Various Temperatures with Particular Reference to Stresses Below the Proportional Limit," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 80 (1928).

cast metal has better resistance to creep than fabricated metal of the same composition.

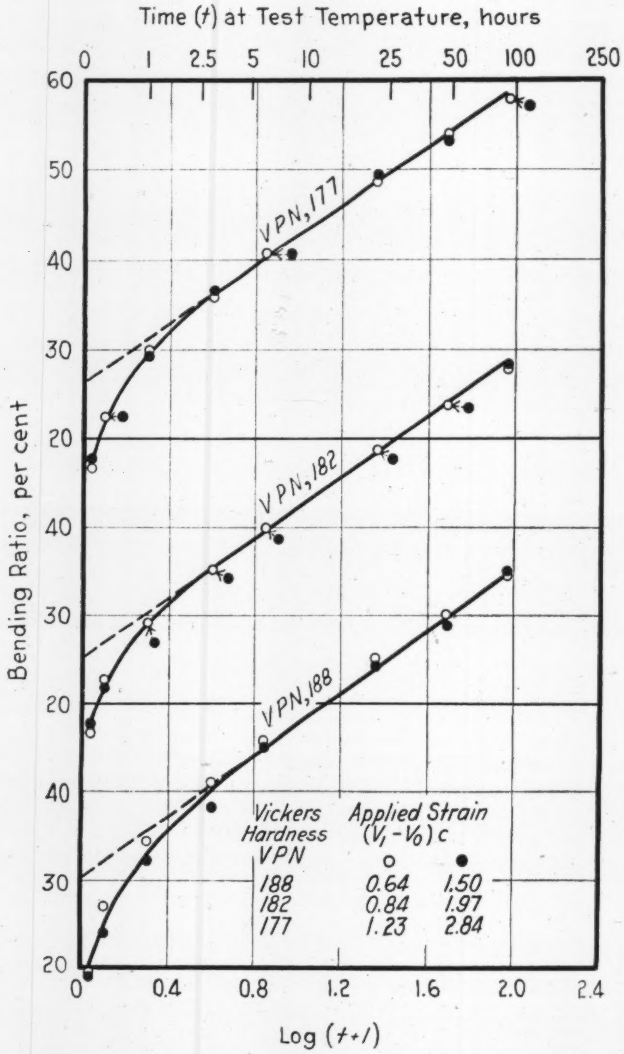

FIG. 15.—Bending Ratio of Cold-Rolled Phosphor Bronze Plotted Against the Logarithm of Time at the Test Temperature of 250° C.

*Time Variable:*

The study of the flow of metals with time under load at elevated temperatures offers an attractive possibility for application of the bend test. With that test the cost of equipment is nominal so the scope of experimental work need not be seriously limited by that factor as it is with the conventional creep test. It is desirable, therefore, to study observations of permanent bending as a function of time and to consider their interpretation in relation to tensile creep test observations.

Moore and Beckinsale collected data on the permanent bending of 70–30 brass over considerable time intervals. Their values of the bending ratio for several different temperatures are plotted in Fig. 13 against the logarithm of time in hours plus one so that the origin corresponds to zero time. The author's results on phosphor bronze tested at 150° C. are plotted in Fig. 14 and on the same material tested at 250° C. in Fig. 15. All three figures show that the greatest reduction in rate of plastic flow occurs within the first hour under test. Consequently, for tests with time constant, the time at test temperature ought to be at least one hour because the effects of the unavoidable variations in effective time at the test temperature are then minimized.

The different hardness numbers identified with the several curves were produced by different degrees of cold reduction. Consequently, from the fact that the harder materials exhibit a greater rate of permanent bending at a given time, it may be concluded that the rate of plastic deformation increases markedly with the amount of previous cold rolling. Thus, the bend test reveals at once the effect of an important variable in the preparation of metals.

The permanent bending curves for time variable show clearly that the bending ratio varies linearly with the logarithm of time after the first few hours and up to at least 250 hours at the test temperature practically independently of the two applied strain values used when the bending ratio is not over 50 or 60 per cent. Consequently, the bending ratio may be expressed within those limitations by the simple relation:

$$\frac{(V_3 - V_0)}{(V_1 - V_0)} = A + B \log (t + 1) \qquad (7)$$

where $A$ and $B$ are constants determined by the intercept and slope, respectively, of the linear portion of the curve and $t$ is the time at the test temperature.

Comparison of the bend and tensile creep test on the basis of net creep involves complications except when the plastic deformation is very small because of the change in load with plastic flow in the bend test. That complication can be avoided by basing the comparison on the instantaneous rate of creep at a specified time. Differentiating, Eq. 7 gives the instantaneous rate of creep in terms of bend test observations:

$$\frac{d(V_3 - V_0)c}{dt} = \frac{0.434\, B(V_1 - V_0)c}{(t+1)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Here again $(V_3 - V_0)c$ is the apparent plastic deformation, $(V_1 - V_0)c$ the applied strain, and $t$ the time at the test temperature in hours. It remains to evaluate the bend test load in stress units. The close correspondence between the curves of bending ratio *versus* time taken with widely different values of applied strain show that the plastic deformation is proportional to the load under the experimental conditions. That is to say, the plastic deformation is proportional to the distance from the neutral axis or very nearly so. Consequently, the maximum stress $p_m$ is given by:

$$p_m = (V_1 - V_3)c\,E \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

where $E$ is the true elastic modulus at the test temperature. It corresponds with the steady creep load of tension testing. By the same token, the apparent plastic strain is numerically equal to the plastic deformation occurring under the stress $p_m$. Since the elastic component of tensile creep at a constant load and temperature is independent of time, the rate of flow measured by tensile creep tests corresponds with that given by Eq. 8. Thus the results of bending and tensile creep tests are directly comparable under experimental conditions which eliminate effects of difference in size of the test specimens.

TABLE II.—CALCULATION OF INSTANTANEOUS LOAD AND RATE OF PLASTIC DEFORMATION FROM VARIATION OF BENDING RATIO WITH TIME.

Material: 2 per cent tin bronze cold-rolled and tempered 300° C. Thickness $2c = 0.015$ in.

| Time ($t$), hours | Bending Ratio, $\frac{V_3 - V_0}{V_1 - V_0}$ | Applied Strain, $(V_1 - V_0)c$ | Apparent Plastic Strain, $(V_3 - V_0)c$ | Maximum Stress ($p_m$), lb. per sq. in. | Rate of Plastic Deformation, $\frac{d(V_3 - V_0)c}{dt}$ |
|---|---|---|---|---|---|
| TEST TEMPERATURE 150° C., CONSTANT $B = 0.058$, MODULUS OF ELASTICITY = 14,200,000 | | | | | |
| 5 | 0.052 | 0.00062 | 0.000032 | 8 400 | 0.0000026 |
| 5 | 0.052 | 0.00151 | 0.000078 | 20 300 | 0.0000063 |
| 50 | 0.105 | 0.00062 | 0.000065 | 7 800 | 0.00000031 |
| 50 | 0.105 | 0.00151 | 0.000158 | 19 200 | 0.00000075 |
| TEST TEMPERATURE 250° C., CONSTANT $B = 0.173$, MODULUS OF ELASTICITY = 14,000,000 | | | | | |
| 5 | 0.44 | 0.00064 | 0.00028 | 5 000 | 0.0000079 |
| 5 | 0.44 | 0.00150 | 0.00066 | 11 800 | 0.0000188 |
| 50 | 0.60 | 0.00064 | 0.00038 | 3 600 | 0.0000009 |
| 50 | 0.60 | 0.00150 | 0.00090 | 8 400 | 0.0000022 |
| 5 | 0.44 | 0.00107 | 0.00047 | 8 400 | 0.0000134 |

The application of the foregoing relations is illustrated by Table II where values of the maximum stress and of the rate of plastic deformation in phosphor bronze, calculated from the bending ratio and applied strain at particular times, are given. The values used for true modulus of elasticity were estimated from the known value at room temperature and the modulus-temperature curve of Fig. 11. With the test temperature of 150° C., the bending ratio is small so the decrease in rate of plastic deformation with time at a substantially constant stress can be seen at once. At 250° C., however, the bending ratio increases rapidly with time so the maximum stress can no longer be considered constant. The effect of time at a constant stress may nevertheless be determined readily by calculating a value of applied strain $(V_1 - V_0)c$, at the time in question from the stress found at the base time. If the value of that applied strain falls within the range of experimental values, a reliable value of the corresponding rate of plastic deformation can be calculated with it as in the bottom row of Table II. Thus, the rate of plastic deformation as a function of time at constant load can be calculated by a simple interpolation from bend test observations when the bending ratio is independent of the applied strain.

TABLE III.—BENDING CONSTANTS OF PHOSPHOR-BRONZE RIBBON AS COLD-ROLLED TO 0.015 IN. THICK AND TEMPERED AT 300° C. FOR 15 MINUTES.

| Test Temperature, deg. Cent. | Bending Ratio[a] | Derived Constants | |
|---|---|---|---|
| | | A | B |
| 150 | 3.0 | 0.008 | 0.058 |
| 250 | 26.0 | 0.302 | 0.173 |

[a] Time, $t$, = 15 minutes. $(V_1 - V_0)c = 0.002$.

The foregoing considerations point to the constant $B$ of Eq. 7 as a basic characteristic of a metal. The constant $A$ is likewise a significant property, for it measures the initial set. These two constants express quite completely the flow characteristics of a metal at a given temperature under nominal restrictions, so their variation with temperature is a matter of greater interest than that of the bending ratio at constant applied strain. The test results are insufficiently extensive to permit drawing such a curve, but the trend of the relation may be seen from Table III. The increase in $A$ with 100° C. rise in test temperature is very much greater than that in $B$. In other words, the initial set increases more rapidly than the rate of plastic deformation. The value of the bending ratio changes in a degree intermediate between that of the constants suggesting that both constants contribute to its value. The data presented here justify confidence in the ability of the bend test to supplement the tensile creep test in the investigation of flow in metals at high temperatures.

## SUMMARY AND CONCLUSIONS

In recognition of the need for a simple and inexpensive test method for studying the quasi-elastic characteristics of metals at elevated temperatures, the method of Moore and Beckinsale was elaborated. The method has the distinctive advantages that strain observations are made at room temperature, that the test temperature is not restricted to the range of useful strength of available alloys, and that very minute plastic deformations can be detected and measured with considerable accuracy by means of simple and inexpensive apparatus. Particular attention was paid to conditions producing very small plastic deformations, which field has not previously been explored.

The curvature of strip specimens is measured before and after heating to the test temperature while constrained to a uniform known curvature. Test observations are recorded as apparent plastic strain (change in curvature of the specimen times its half thickness) and as applied strain (curvature of the specimen as constrained times its half thickness). Both these variables are components of the stress-strain curve and may be calculated from it.

The curve obtained by plotting the bending ratio (apparent plastic strain in per cent of applied strain) against applied strain swings upward when the applied strain in the extreme fibers exceeds the value corresponding to the proportional limit in the tension test. That value of applied strain is called the relaxation strain to avoid implications as to the shape of the true modulus curve. Tests at moderate temperatures indicate that it is a basic property of the metal tested corresponding to the so-called "proportional limit." At higher temperatures the correspondence disappears.

Notable plastic deformation occurs below the relaxation strain at room temperatures and rapidly increases in relative magnitude as the test temperature is increased. This fact has already been demonstrated by creep tests, but the plastic component is not so clearly isolated with that test. The plastic deformation below the relaxation strain is usually proportional to the applied strain or very nearly so. This fact greatly simplifies interpretation of the bending observations because the apparent plastic strain has then a value equal to the true plastic deformation, otherwise its value is lower. Evidently part of the decrease in apparent modulus with temperature as measured by the usual tension test is caused by this "proportional plasticity."

A critical temperature representative of the temperature range of useful strength in metals can be readily determined by the bend test. It is identified by the term "relaxation temperature" and defined as that temperature at which the bending ratio reaches 10 per cent for a specified applied strain and time at the test temperature.

When the bending ratio of phosphor bronze is plotted against the logarithm of time, a linear curve is obtained after the first few hours provided the bending ratio is small. From the linear portion of that curve the rate of plastic deformation and maximum stress at a particular time can be easily calculated. Those values are directly comparable with the creep rate and load of tension creep testing. The curves show that initial set increases more rapidly with temperature than does the rate of plastic deformation and confirm the previous finding that cold working predisposes to creep.

*Acknowledgment:*

The work here described was commenced at the suggestion of Mr. P. H. Brace who has maintained a close and helpful interest in the work. The writer is indebted to A. Nadai and G. B. Karelitz for technical advice and much credit is due Messrs. R. W. Carson, J. S. Newton and W. N. Meyer, at the time Westinghouse graduate students, who secured practically all the test data. This paper is published with the approval of the Management of the Westinghouse Electric and Manufacturing Co. as a contribution from their Research Laboratories.

## APPENDIX

### ESTIMATION OF RESIDUAL CURVATURE PRODUCED BY UNIFORM BENDING OF A STRIP FROM ITS STRESS-STRAIN DIAGRAM

Let $OST$ of Fig. 16 be the stress-strain curve of the metal. For the desired value of applied strain, $\varepsilon_1$, draw the vertical $\varepsilon_1 p_1$, select arbitrarily two points, $p'$ and $p''$ on that vertical between $p_1$ and $p_2$ and connect them with origin by straight lines. Mark off 10 equal strain increments on abscissæ between $O$ and $\varepsilon_1$ and tabulate vertical differences between $Op'$ and $OST$ for each of those strain values, indicating values above *OST* by positive sign and those below by negative sign and multiply by corresponding strain values. Carry out the same tabulation for differences between $Op''$ and *OST*. Take the algebraic sum of the products after halving those for $\varepsilon_1$ and identify the values for $Op'$ by $\Sigma M'$ and those for $Op''$ by $\Sigma M''$. Then the apparent value of stress lost on release, $p_3$, is given by:

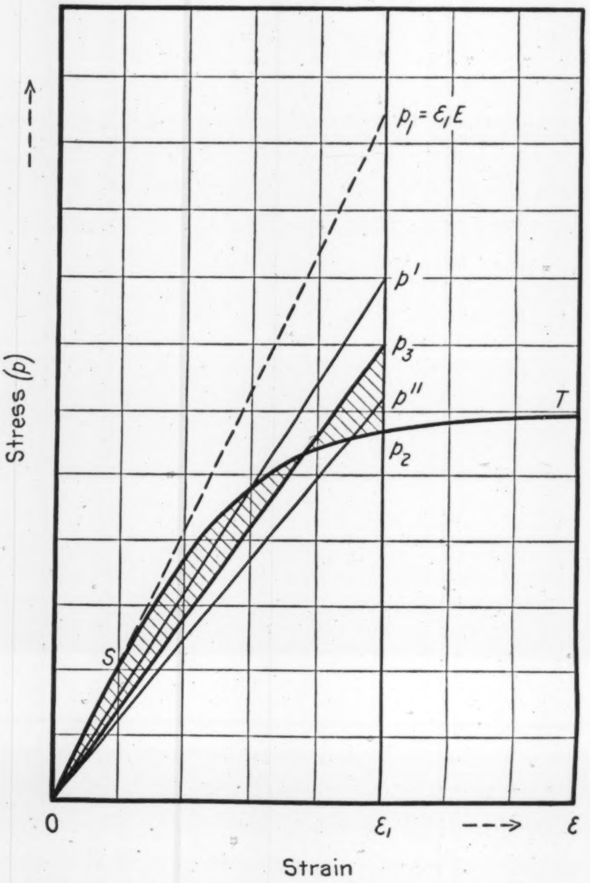

FIG. 16.—Stress-Strain Curve.

$$p_3 = p' - \frac{\Sigma M'}{\Sigma M' - \Sigma M''}(p' - p'')$$

From Eq. 2, the residual curvature is then:

$$V_3 = \frac{p_1 - p_3}{Ec}$$

# DAMPING CAPACITY OF MATERIALS

By G. S. von Heydekampf[1]

## Synopsis

The damping capacity of materials is defined, and shown to be identical with the "mechanical hysteresis effect" or "internal friction of solids." Different methods of determining the damping capacity are described. Results are given of damping tests obtained in torsion and flexure at the Woehler Institute, Braunschweig.

The significance of this property is explained. In the author's opinion, the damping capacity is not only important in cases of direct application, such as resonance vibrations, but deserves attention as a novel method of investigating the elastic behavior in the range of low stresses. Recent tests by Professor Foeppl indicate that the damping capacity is a measure of "dynamic ductility" which should be coordinated with the fatigue limit in the same way that tensile strength and elongation are considered together.

## Definition

Engineering materials, particularly metals, are able to dissipate energy without failure when subjected to cyclic stresses below the fatigue limit. The effect in question has been studied by many investigators (see appended bibliography). This property is known as "mechanical hysteresis effect," "internal friction" (of solids), or "damping capacity." The latter nomenclature was proposed by O. Foeppl in 1923, who offered the following definition:

"Damping Capacity" is the amount of work dissipated into heat by a unit volume of the material during a completely reversed cycle of unit stress.

This damping capacity is measured in inch-pounds per cubic inch per cycle. It will be noted that this is the dimension of a stress per cycle. The conversion factor to obtain inch-pounds per cubic inch from centimeter-kilogram per cubic centimeter, which dimension is frequently used in the literature, is 14.2.

The damping capacity is given as work in absolute measure. For comparison between materials of different moduli of elasticity the "specific" or "proportional" damping capacity is a better measure, which is obtained by dividing the damping capacity by the potential energy accumulated at the maximum stress of the cycle considered. In accordance with the theory of elasticity, we have, in terms of work per unit volume:

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$$A = \frac{S^2}{2E}$$

where $A$ = potential energy at maximum stress;
$S$ = maximum stress; and
$E$ = modulus of elasticity.

Specific damping capacity may be expressed as follows:

$$p = \frac{d}{A}$$

where $d$ = damping capacity as an absolute measure of work; and
$p$ = specific damping capacity.

It will be seen that $p$ is a dimensionless quantity. It can also be shown that $p$ equals twice the logarithmic decrement of a damped vibration (see Damping or Free Vibration Method below). This as well as the decrement is a useful measure for the damping capacity while $d$ should only be applied in the case of materials of the same modulus of elasticity.

Usually, the damping capacity is determined from solid round specimens. The values of $d$ or $p$ obtained in this way represent the average damping values over the entire cross-section. This is the correct damping capacity, as defined above, if the stress is the same in all fibers of the cross-section, for example in tension-compression tests. In flexure and torsion tests, however, the stress is not uniformly distributed, and each fiber contributes an individual damping value to the total amount which is measured as the average. In order to obtain absolute damping values in these cases, the average damping values can be converted into fiber damping values by means of computation (**19, 34, 36**).[1] It is also possible to obtain fiber damping values from flexure or torsion tests by the use of hollow specimens which have practically a uniform stress distribution, if the wall thickness is small compared to the diameter. Ordinarily, the results are given as average over the circular cross-section. Only when results of tests with different types of cross-section (circular and rectangular) have to be compared, the fiber values must be used, appropriately designated by a subscript.

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[1] The boldface numbers in parentheses refer to the reports and papers given in the list of references appended to this paper, see p. 170.

## DETERMINATION OF THE DAMPING CAPACITY

*Static Method* (Hysteresis test) **(1, 5, 6, 7, 8, 9, 10, 25, 26, 27, 34, 36, 37):**

In the case of a stress cycle on a reversible static testing device the loading and the unloading curves in the stress-strain diagram deviate from each other, provided the strain gage used be sufficiently sensitive. In order to detect such deviations in low stress ranges, very accurate loading has to be applied, which is practically impossible except by dead weights. It is for this reason that the hysteresis method has been used mostly with torsion or bending stresses rather than tensile stresses. The deformation produced by comparatively small dead weights is much greater in torsion or bending than in tension. Consequently most of the experimental data given in this paper were obtained in torsion or bending, while all general statements cover any kind of stress unless otherwise noted.

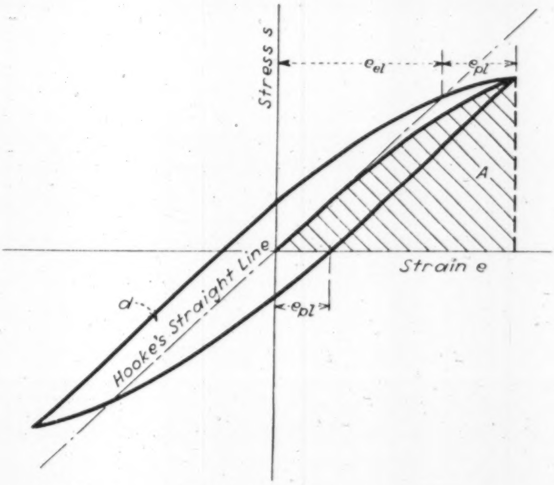

FIG. 1.—Schematic Hysteresis Loop for a Completely Reversed Cycle of Stress.

The stress-strain diagram for a complete cycle of stress forms the so-called hysteresis loop the area of which represents the work absorbed, $d$ (Fig. 1). The area under the loading line from zero to the maximum stress in the same diagram is a measure of the potential energy, $A$, which is accumulated due to elastic deformation. The static loading test in its highly refined form as referred to herein therefore furnishes either $d$ or $p$, the latter being the quotient of $d$ and $A$.

The rate of loading does not affect the amount of work absorbed as might be expected. A cycle on a static machine will require about 5 minutes or more, so that the rate of loading is one-fifth per minute or less. Becker (26) compared the hysteresis determined in this way with the one at speeds up to 2600 cycles per minute, using different methods to be described below. His results show the independence of the damping capacity from the rate of loading (see also Reference (5)).

Autographic records of the hysteresis loop in tension and compression have been made, but such apparatus is exceedingly delicate and cannot be built cheaply, and, so far, testing practice has been to use sensitive strain gages and to plot the results point by point.

It should be borne in mind that in this test only the minute differences in the loading and unloading curve are important, whereas

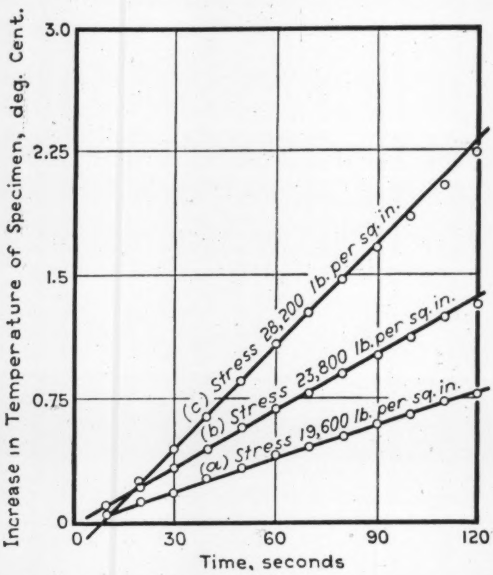

FIG. 2.—Starting Curves for Specimen 95V (Steel), Taken on Torsional Fatigue Testing Machine, Foeppl-Busemann ($n$ = 860 r.p..m).

even in exact determinations of the proportional limit an average value is sufficient. The accuracy of an ordinary strainometer or of a standard testing machine is hence not sufficient for this purpose. Here we must measure such small amounts that in ordinary stress-strain curves they would hardly be larger than the experimental error.

*Starting Test* (26, 34, 36, 37):

If a fatigue testing machine is available which permits accurate determination of the temperature of the specimen, the so-called "starting test" furnishes the damping values in a very convenient manner.

The damping energy is continuously converted into heat. So when applying repeated cycles of stress on a fatigue machine, the temperature of the specimen must be raised due to the internal heat development. As a matter of fact, this heating effect can be observed beginning from very small stresses far below the fatigue limit, unless most of the heat is immediately carried away to the grips and adjacent machine parts. Unfortunately, this is the case with most of the American fatigue testing machines, using short specimens in which only a small volume of the material is subjected to the maximum stress. In such cases the total amount of heat generated is too small to produce an appreciable change of temperature and the methods mentioned below fail. Such temperature methods are, however, readily applicable for long gage length specimens, say 8 in. by ½-in. diameter, particularly in connection with non-rotating bending, tension, or torsion fatigue testing machines, which are more suitable for thermocouple temperature measurements than the ordinary rotating fatigue specimens. The accuracy of temperature measurements required with these long specimens is about 0.05° C.

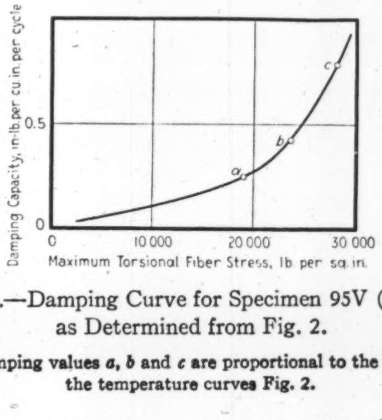

FIG. 3.—Damping Curve for Specimen 95V (Steel), as Determined from Fig. 2.

**The damping values *a*, *b* and *c* are proportional to the slope of the temperature curves Fig. 2.**

The "starting test" for determining damping capacity consists in reading the temperature as a function of time elapsed after starting the machine. Curves of such kind are shown in Fig. 2 for a steel at three different fiber stresses. This particular test was made in torsion and a straight initial portion of the curves is evident. It can be proved that the slope of this straight part is proportional to the damping capacity. It is only necessary to know the specific heat of the material under investigation in order to evaluate curves like those in Fig. 2 into damping values. This has been done in Fig. 3, where the three damping capacity values *a*, *b*, and *c* of Fig. 2 have been plotted against the maximum fiber stresses. In this way, the damping capacity can be determined for any fiber stress desired. It is of particular advantage that the external conditions of temperature and heat transfer do not impair the results. It is only the length of the straight portion of the starting curve, not its slope, that may change with varying external circumstances.

*Equilibrium Temperature Method* (3, 4, 17, 23, 26, 28, 30, 33, 34):

When the starting test is continued to become a real fatigue test, the temperature of the specimen increases in accordance with the scheme given in Fig. 4. While at the beginning practically all the heat generated is utilized to raise the specimen's temperature (period 1 in Fig. 4), gradually more and more heat is transferred to the surrounding air and machine parts, until after a period of about one hour or more the heat generation just balances the amount of heat carried away by convexion and radiation. The temperature of the specimen no longer increases; a state of equilibrium prevails if nothing disturbs the test (period 2 in Fig. 4). The absolute amount of the equilibrium temperature, $t_e$, can vary with the room temperature and other conditions of test. Although the value $t_e$ is proportional to the damping capacity, it is necessary to determine the factor of proportionality every time, when $t_e$ has been measured. In contrast to the starting test, in this test it is not allowable to take the temperature of a fatigue specimen alone as a measure of its damping capacity, without giving data as to the external condition. The factor of proportionality is easy to determine by stopping the machine and watching the drop of temperature when the internal generation of heat ceases. This furnishes a curve similar to the last part of Fig. 4 (period 3). The details of the further evaluation cannot be given here, and is not necessary since similar methods are familiar to electrical engineers in

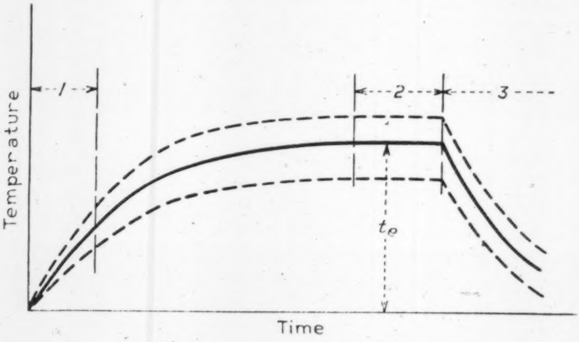

FIG. 4.—Schematic Diagram Showing Temperature of a Fatigue Specimen as a Function of Time Elapsed After Starting the Machine.

The dotted diagrams are obtained with different stresses.

measurements of losses of electrical generators and motors. It may be sufficient to state that the combined equilibrium and stopping tests permit finding the damping capacity in a very accurate though not in so convenient a way. The accuracy is high, since the equilibrating temperature is the result of thousands of cycles, representing an integral value with small individual error.

*Energy Input Method* (20, 24, 26):

The damping capacity can also be determined from the energy input into a fatigue specimen. Whatever the type of machine used, it is in most cases very difficult to separate the energy necessary to overcome frictional and other losses in the machine from the energy absorbed by the specimen itself. It is for this reason that the energy input method is a rough check on other methods rather than a primary standard way of determining the damping capacity.

*Damping or Free Vibration Method* (2, 5, 12, 13, 15, 21, 23, 24, 27, 29, 31, 32, 34, 36):

While all methods discussed above make use of machines or apparatus which originally were not intended to measure the damping capacity but primarily serve other purposes, the group of instruments mentioned in this paragraph is designed solely for investigations of damping qualities. The fundamental idea of these "damping" machines is that the specimen performs free vibrations which are damped only by internal friction of the material. The situation is somewhat similar to the one mentioned under Energy Input Method; the principle is simple, but possible external losses are liable to make the results highly erratic, as has been shown by Foeppl, Pertz and Knackstedt. Results with this method can only be relied upon if accurate checking methods are available, but once calibrated this free vibration method is far superior to all other methods described, because the whole damping characteristic from zero stress to the yield point can be determined with a single test usually requiring less than 20 minutes. So far as the author knows, there is only the damping tester Foeppl-Pertz on the market, which has gone through the mill of thorough investigation of its accuracy and limits. This torsional machine has been briefly described in the February, 1931, issues of *Metals and Alloys* and *Instruments*. A more detailed discussion will appear in a paper to be presented before the American Society of Mechanical Engineers (December, 1931.)

*Lateral Deflection Method* (16):

We have discussed the damping capacity as a physical fact without hinting at its significance. The last method of determination to be mentioned gives an opportunity to show one field of direct practical application.

A. L. Kimball has investigated the influence of damping capacity or, in his terms, of internal friction of solids, on the behavior of rotating shafts. Among other facts, he discovered that a rotating cantilever shaft loaded at its free end by a weight does not deflect in the vertical plane of the weight, as the classical theory of elasticity would lead one to expect. Such a revolving shaft shows a lateral deflection propor-

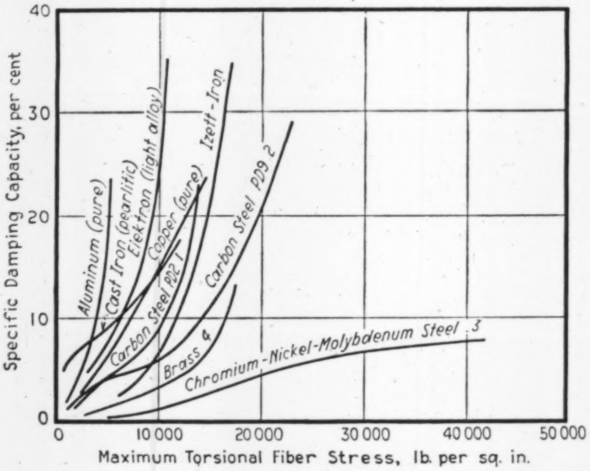

FIG. 5.—Damping Characteristics for Materials of Comparatively High Damping Capacity.

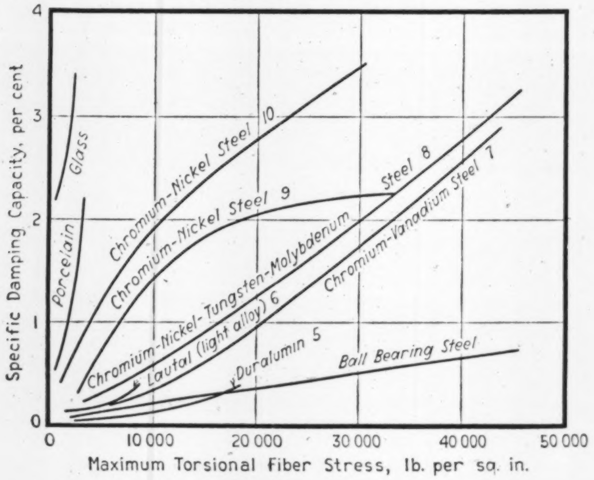

FIG. 6.—Damping Characteristics for Materials of Comparatively Low Damping Capacity.

## TABLE I.—COMPOSITIONS OF MATERIALS TESTED.

STEELS

| | | Material | Carbon, per cent | Manganese, per cent | Silicon, per cent | Chromium, per cent | Nickel, per cent | Tungsten, per cent | Molybdenum, per cent | Vanadium, per cent | Phosphorus, per cent | Sulfur, per cent |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| No. 95 V | Figs. 2 and 3 | Manganese Steel | 0.52 | 1.32 | 0.22 | .... | .... | ... | .... | .... | 0.029 | 0.015 |
| No. 1 | Fig. 5 | Carbon Steel PD2 | 0.22 | 0.04 | 0.07 | .... | .... | ... | .... | .... | 0.01 | 0.014 |
| No. 2 | Fig. 5 | Carbon Steel PD9 | 0.93 | 0.10 | 0.11 | .... | .... | ... | .... | .... | 0.01 | 0.015 |
| No. 3 | Fig. 5 | Chromium-Nickel-Molybdenum Steel | 0.16 | 0.40 | 0.15 | 1.90 | 4.90 | ... | 0.70 | .... | .... | .... |
| No. 7 | Fig. 6 | Chromium-Vanadium Steel | 0.5 | 0.6 | 0.2 | 1.05 | .... | ... | .... | 0.17 | .... | .... |
| No. 8 | Fig. 6 | Chromium-Nickel-Tungsten-Molybdenum Steel | 0.4 | 0.45 | 0.3 | 1.15 | 2.5 | 0.9 | 0.25 | .... | .... | .... |
| No. 9 | Fig. 6 | Chromium-Nickel Steel | 0.3 | .... | .... | 0.95 | 4.2 | ... | .... | .... | .... | .... |
| No. 10 | Fig. 6 | Chromium-Nickel Steel | 0.3 | .... | .... | 0.95 | 4.2 | ... | .... | .... | .... | .... |
| No. 11 | Fig. 7 | Carbon Steel | 0.06 | 0.42 | .... | .... | .... | ... | .... | .... | 0.052 | 0.03 |
| No. 12 | Fig. 7 | Carbon Steel | 0.35 | 0.76 | 0.28 | .... | .... | ... | .... | .... | 0.011 | 0.026 |
| No. 13 | Fig. 7 | Chromium-Nickel Steel | 0.29 | 1.57 | 0.25 | 0.18 | 0.41 | ... | .... | .... | 0.023 | 0.016 |
| No. 14 | Fig. 7 | Chromium-Nickel Steel | 0.36 | 0.85 | 0.29 | 1 4 | 0.38 | ... | .... | .... | 0.01 | 0.016 |

NON-FERROUS METALS

| | | Material | Copper, per cent | Aluminum, per cent | Silicon, per cent | Lead, per cent | Zinc, per cent | Magnesium, per cent | Manganese, per cent |
|---|---|---|---|---|---|---|---|---|---|
| No. 4 | Fig. 5 | Brass | 57.5–58.5 | .... | ... | 2.5–2.9 | 40–41.4 | ... | ........ |
| No. 5 | Fig. 6 | Duralumin Leg 681B aged | 3.5–4.5 | Bal. | ... | ....... | ....... | 0.5 | 0.25–1.0 |
| No. 6 | Fig. 6 | Lautal | 4.0 | 94.0 | 2.0 | ....... | ....... | ... | ........ |

## TABLE II.—PHYSICAL PROPERTIES OF MATERIALS TESTED.

| | | Material | Yield Point, lb. per sq. in. | Tensile Strength, lb. per sq. in. | Elongation per cent | Reduction of Area, per cent | Fatigue Limit, lb. per sq. in. | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | Flexure | Torsion |
| No. 95 V | Figs. 2 and 3 | Manganese Steel | 75 000 | 105 000 | 22.4 | 64.0 | ..... | 31 300 |
| No. 1 | Fig. 5 | Carbon-Steel PD2 | ...... | ...... | .... | .... | ..... | ..... |
| No. 2 | Fig. 5 | Carbon-Steel PD9 | ...... | ...... | .... | .... | 37 000 | ..... |
| No. 3 | Fig. 5 | Chromium-Nickel-Molybdenum Steel | ...... | 190 000 | 11[a] | .... | 77 000 | ..... |
| No. 4 | Fig. 5 | Brass | ...... | 62 000 | 25[a] | .... | 28 500 | ..... |
| No. 5 | Fig. 6 | Duralumin Leg 681 B aged | 38 400 | 57 000 | 19[a] | 24 | 20 000 | 12 800 |
| No. 6 | Fig. 6 | Lautal | ...... | 57 000 | 26[a] | .... | 21 000 | ..... |
| No. 7 | Fig. 6 | Chromium-Vanadium Steel | ...... | 178 000 | 8.5[a] | .... | 87 000 | ..... |
| No. 8 | Fig. 6 | Chromium-Nickel-Tungsten-Molybdenum Steel | ...... | 175 000 | 11.5[a] | .... | 78 000 | ..... |
| No. 9 | Fig. 6 | Chromium-Nickel Steel | 92 500 to 114 000 | 120 000 to 142 000 | 20 to 16 | 75 to 60 | ..... | ..... |
| No. 10 | Fig. 6 | Chromium-Nickel Steel | 120 000 to 142 000 | 142 000 to 164 000 | 16 to 12 | 65 to 50 | ..... | ..... |
| No. 11 | Fig. 7 | Carbon Steel | 41 000 | 53 000 | 28.5 | .... | 36 000 | 23 900 |
| No. 12 | Fig. 7 | Carbon Steel | 43 000 | 76 000 | 27.8 | 52 | 37 500 | 23 600 |
| No 13 | Fig. 7 | Chromium-Nickel Steel | 89 000 | 109 000 | 21.3 | 67 | 56 000 | 29 600 |
| No. 14 | Fig. 7 | Chromium-Nickel Steel | 117 000 | 129 000 | 18.5 | 61 | 62 000 | 42 000 |

[a] Gage length equal to ten times diameter.

tional to the damping capacity of the shaft material. Furthermore, it has been shown by Kimball, that the damping capacity can cause undesirable whirling effects so that a knowledge of internal friction of shaft materials seems imperative from this point of view.

## SOME RESULTS OF DAMPING TESTS

In Fig. 3, a typical damping curve for a steel is given. There the absolute damping capacity *d* is plotted against the torsional fiber stress. In Figs. 5 and 6, the characteristics for various materials are shown, using the proportional damping capacity for comparison. The scale for the damping values in Fig. 6 is ten times as large as that in Fig. 5. Some additional chemical and physical data on the materials represented by Figs. 5 and 6 are listed in Tables I and II. All these tests were made in the Woehler Institute, Braunschweig.

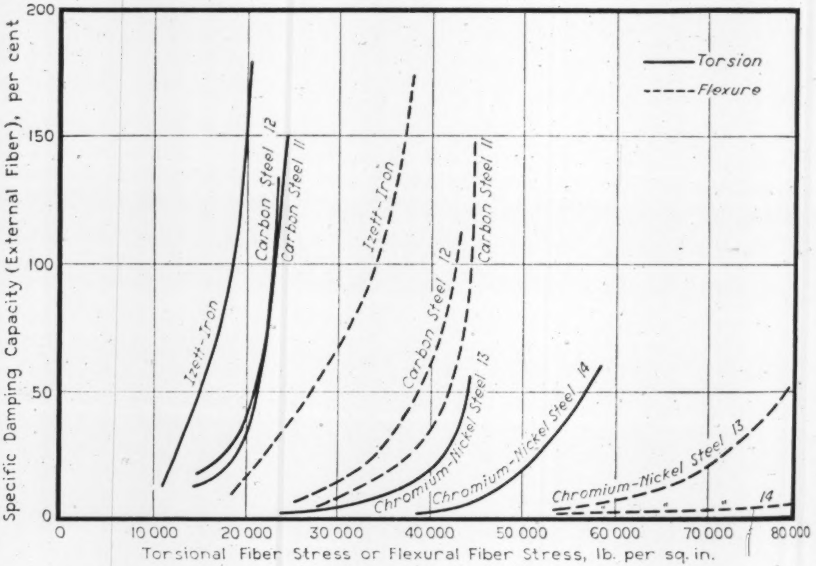

FIG. 7.—Comparison of Damping Values in Torsion and Flexure.

It is important to realize that the damping capacity is a physical property, independent of the method of determination or size of specimen or of the type of machine used. Figures 5 and 6 indicate that the damping capacity is primarily a function of the stress imposed. Other influences such as overstressing are not discussed in this paper.

Although it seems proper to coordinate higher damping capacity with smaller tensile (or hardness) values, at least so far as steels are concerned, many experiments have shown that the damping capacity may be quite different for steels of identical chemical and tensile properties. This discrepancy between tensile properties and damping values is still more distinctive in the case of non-ferrous metals and non-metals. It is particularly interesting to note the extremely small damping values of certain light alloys (duralumin, Lautal) while pure aluminum and Elektron metal display much higher values. On the other hand, one finds a surprisingly high damping capacity in cast iron.

In Fig. 7, a comparison is made as to damping with flexural and with torsional stresses. Apparently the shear stresses involved in the torsional test are responsible for the major part of the damping effect, because the steep rise of the damping curve starts much earlier in torsion than in flexure. The curves as a whole are, however, similar in both cases so that the preliminary theory has been proposed that also in flexure, the shear stresses, occurring under an angle of 45 deg. to the main axis of the specimen, are the cause of the damping effect (37). It is easier to measure the high damping values in torsion, and consequently most data on the subject, published so far, are in connection with tests in torsion.

## SIGNIFICANCE OF THE DAMPING CAPACITY

While in rotating shafts a high damping capacity may cause undesired whirling effects, the same damping capacity is beneficial in torsional oscillations of crankshafts or other freely vibrating parts. If such critical, or resonance, impulses occur for instance in a combustion motor, the crankshaft, if made from a highly damping steel, is restricted to small amplitudes, since the energy of the impulses is immediately dissipated by internal friction. The resulting small amplitude of the oscillation involves stresses which may be well within the fatigue limit of the particular steel. If the same motor would be equipped with a crankshaft of low damping capacity, high resonance stresses exceeding the fatigue range will be more likely to occur since there is no self protection by internal damping of the material. It is still an open question whether the internal effect of the shaft material alone will absolutely be sufficient to prevent resonance failures. But the designer of a machine part liable to suffer vibrations will certainly choose a steel having higher damping capacity, all other qualities being the same.

In these instances of immediate value for vibration problems, the damping capacity has to be studied in close relation to the fatigue properties. It should be emphasized that the results obtained at the Woehler Institute clearly show that there is no reliable way to predetermine the true endurance limit by damping tests (25, 30, 34). The curves in Figs. 3, 5, 6, and 7 do not show any "knee" or other irregular behavior indicating any point such as the fatigue limit. In other words, damping tests or, what is the same thing, "rise of temperature tests" are not reliable short-time fatigue methods. They do not replace but rather supplement the fatigue tests, furnishing additional information which cannot be easily obtained by other tests.

On the other hand, there is a relationship between the damping capacity and the static stress-strain diagram since "hysteresis" tests are one method used to find the damping values. Let us write, with reference to Fig. 1, $\lambda = \frac{e_{pl}}{e_{el}}$ where $e_{el}$ represents the deformation which would occur in accordance with Hooke's law. The value $e_{pl}$ is the "plastic" deformation, or in other words, the deviation of the actual stress-strain diagram from the theoretical straight line (Hooke's law of proportionality). Within a certain range of stress, normally coinciding with the fatigue range, $e_{pl}$ equals half the width of the hysteresis loop. It has been found by numerous torsional experiments of O. Foeppl (19, 22, 25, 34), that for steels the approximate equation $\lambda = 0.2\ p$ obtains where the factor 0.2 is an average value and may vary from 0.18 to 0.23. The damping curve is therefore only a kind of stress-strain diagram, since the latter can be plotted from the former. The accuracy of the damping curves is however much higher than that of the usual static stress-strain diagrams. If we take for instance a steel similar to No. 7 in Fig. 6, we find $p = 0.6$ per cent at a stress of 15,000 lb. per sq. in. Then the plastic deformation would be

$$e_{pl} = e_{el} \times \lambda = e_{el} \times 0.2\ p = \frac{S}{E} \times 0.2\ p =$$

$$\frac{15{,}000 \times 0.2 \times 0.006}{12{,}000{,}000} = 0.0000015$$

It is very difficult to determine such small amounts of nonelastic deformation by means of ordinary strain measurements, because even if an extensometer of sufficient accuracy is available, the accuracy of the loading system of the present standard testing machines would hardly be high enough. Damping tests however readily permit the determination of such small values of $p$ and the damping curve should therefore be used as a modified stress-strain diagram for small stresses. This is in accordance with the trend towards higher accuracy in determining the so-called proportional limit. The damping method seems to furnish a novel and promising tool for investigating the range of low stresses which is the one mostly used in service.

From the smooth curves of Figs. 5 and 6, it is obvious that there is no "actual" proportional limit, for if this were true, the values of $p$

should be zero up to the "proportional limit." Such a threshold value does not really exist for any material, not even for porcelain, glass or extremely brittle and hard steel. The damping tests confirm once more the fact that Hook's law is only an approximation. Consequently, O. Foeppl has proposed to no longer use the "elastic limit" but to measure and specify that amount of damping or non-elastic deformation which occurs at the endurance limit. It has been proved beyond doubt that even with billions of cycles, the energy absorption or damping effect goes on without impairing the material or being accompanied by a starting failure. So the fatigue limit and the corresponding damping value, which may be called the damping limit, should be coordinated in a way similar to tensile strength and elongation. "Damping limit" and "elongation" both measure plastic behavior, while fatigue limit and tensile strength give stress limits in the respective ranges.

From this analogy the name "dynamic ductility" for the damping capacity will be understood, while the elongation will be differentiated as the "static ductility." This is believed to be one of the most important features of the damping effect. To give an example, duralumin would be a dynamically brittle material (low damping values), while cast iron must be considered as dynamically ductile (high damping values) (Figs. 5 and 6). For these materials, the static ductilities are opposite and cannot account for the experience of fatigue and notch fatigue tests that surface defects have a very high influence on the life of duralumin specimens, whereas cast iron is almost unaffected by accidental imperfections of surface.

Recent tests at the Woehler Institute (34, 35, 38) indicate that at least for the materials tested (various carbon steels, alloy steels, bronze, brass, light alloys, cast iron) the dynamic ductility as measured by the damping capacity seems to be related to the "tenderness" effect. In other words, a material of higher damping capacity is less sensitive to the influence of surface notches or sudden changes in cross-section, which strongly affect the fatigue strength of "dynamically" brittle materials. Materials having high damping values are able to be deformed to a higher degree than Hooke's law predicts without being damaged. In this way, "stress raisers" may be compensated for by internal yielding, the amount of which is measured by the damping capacity.

It has been impossible to give here more than a short introduction to the subject. There may be a relation between the notched bar impact test and the damping effect. It also seems probable that the plastic action, causing the damping effect, may be related to the creep phenomenon.

*Acknowledgment.*—The author is greatly indebted to Prof. O. Foeppl for the permission to publish results obtained at the Woehler Institute. He also wishes to thank Mr. G. Parzich of the Woehler Institute who helped prepare the tables and figures. Finally the author will not fail to express his particular appreciation of the valuable help and advice of Mr. A. V. de Forest.

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# DISCUSSION

MR. A. V. DE FOREST[1].—Damping capacity as a measure of dynamic ductility seems to be a conception with which we are to become more familiar. It explains a number of curious effects we have all known about but never realized exactly what they meant. The idea of tenderness or the reaction to surface damage which different materials possess is very nicely explained by the term dynamic ductility, and I think we are going to revise our thoughts on the fatigue properties of metals rapidly if this damping capacity idea takes root and grows in the engineering profession.

From the metallographic point of view, I have been very much interested in this test as relating to the question of sonims or other disturbances—in steel particularly. We all know that wrought iron has an enormous damping capacity; struck with a hammer it damps at once, relating this test somewhat with Robin's work on the damping capacity of sound in metals. At Wyckoff Drawn Steel Co. we have attempted to apply this test to cold-drawn screw stock and materials that are heavily loaded with sulfides in order to make them cut freely. Such a test can be readily applied and used to attack that very difficult property, the amount and possibly the distribution of non-metallic material. The work is by no means complete, but it shows a very promising start through the medium of this rather novel test.

MR. H. F. MOORE.[2]—I should like to present what may seem an almost fantastic cartoon of what would happen in a member subjected to repeated stress with a material that had no damping capacity whatever. I cannot imagine any such part being put together and subjected to loads without having very high localized stresses. In fact, I cannot imagine such a material being put together and subjected to loads without starting cracks, and if such a material were used, probably none of our machine parts would stand up.

MR. A. I. LIPETZ.[3]—In the forties of the last century Redtenbacher, of Germany, proved mathematically that at certain speeds depending upon the length of the rails a locomotive could not stand up; the stresses in the springs would be so high that the whole structure would go up in the air. As this has never happened, we knew that it was due to the internal friction between the plates in the springs. The present discussion is most interesting because it suggests that in addition to the friction between the spring plates the internal friction in the metal itself may help the damping.

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[2] Professor of Engineering Materials, University of Illinois, Urbana, Ill.

[3] Consulting Engineer, American Locomotive Co., Schenectady, N. Y.

We also know that crankshafts in internal combustion and other engines made of alloy steels have been replaced in many cases by ordinary low-carbon steels, probably because the damping capacity is higher in this class of steel.

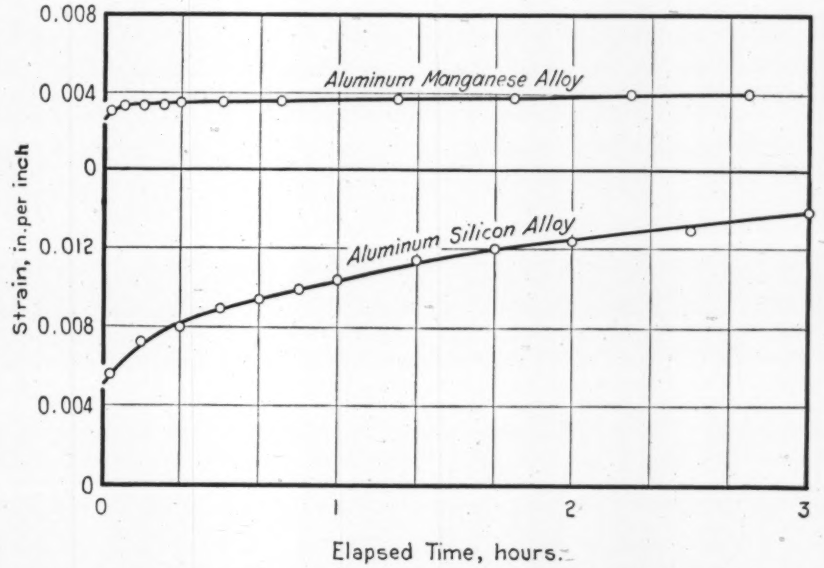

**FIG. 1.—Strain-Time Curves for Two Aluminum Alloys Subjected to Constant Tensile Stress (80 per cent of Ultimate Strength).**

MR. R. G. STURM[1] (*by letter*).—This paper presents a very clear résumé of the present status of research on the damping capacity of materials resulting from internal friction or mechanical hysteresis. As pointed out in the paper, this phenomenon apparently results from minute slippages between the grains or within the crystals of the material. A similar slippage seems to account for the phenomenon of creep, that is, continued elongations under a sustained load.

As a result of more or less experience in determining the creep characteristics of materials, it has been found that this slip phenomenon takes place quite rapidly in the first second after the load is applied, and gradually decreases as time goes on. For materials which have very low creep, the apparent additional strain all occurs in a very short time after the load is applied while in other materials which have a relatively large creep, the slip is not only rapid at the start but continues to be apparent for a long period of time. By plotting the strain under a given load as ordinates against the time of duration of that load as abscissas, one finds a curve which rises more or less rapidly in the vicinity of zero time. The accompanying Fig. 1 shows such curves for two aluminum alloys: aluminum-manganese alloy (1.25 per cent manganese), and aluminum silicon alloy (10 per cent silicon). If the equation of this curve is determined such that the rate of creep at zero time can be found, the amount of plastic strain at any instant after load is applied may be determined. Thus for a case of materials with very low creep, the rate of loading would have quite an appreciable effect upon the plastic creep.

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There seems to be considerable evidence pointing to the fact that the phenomenon of creep, the effect of speed of testing and the damping capacity of materials all arise from the same general behavior of crystals of the material under stress. The author of the paper recognized this in his final paragraph, but it is felt that this apparent relationship is of sufficient importance to deserve more discussion, especially since the damping capacity, rate of creep and effect of speed of testing may eventually all be expressed by a common property of the material. To the writer the term "dynamic ductility" seems to be the most generally applicable to this fundamental phenomenon.

MR. R. E. PETERSON[1] (*by letter*).—An interesting phase of damping measurements mentioned by the author is the possible correlation between damping capacity and susceptibility to stress-concentration effects in fatigue. This latter characteristic has been called "tenderness" by H. F. Moore,[2] who has indicated the seriousness of defects in certain high-strength alloy steels. The designer is vitally interested in the endurance properties with stress concentration effects present, since machine parts are of such a nature as invariably to have stress concentration in the form of holes, fillets, threads, grooves or keyways.

No satisfactory correlation between "tenderness" and any of the ordinary physical properties of materials has been found in the past. Ductility was naturally suggested as a possibility, but it has been shown that no such correlation exists.[1]

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[1] Manager, Mechanics Division, Research Laboratories, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[2] H. F. Moore, "Materials in Machine Construction," *Mechanical Engineering*, October, 1929, p. 739.

Damping capacity, however, gives good possibilities of correlation with "tenderness." Materials having high damping capacity, such as cast iron,[2] show virtually no stress-concentration effect in fatigue.[3]

On the other hand, materials having small damping capacity, such as alloy steels,[3] are known to have large stress concentration effect in fatigue.[4] It seems highly desirable that a systematic series of tests be planned involving a variety of materials with a view to determining the extent of the suggested correlation.

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# ENDURANCE TESTING OF STEEL: COMPARISON OF RESULTS OBTAINED WITH ROTATING-BEAM *VERSUS* AXIALLY-LOADED SPECIMENS[1]

By R. D. France[2]

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## Synopsis

One phase of the study of rail steel at the U. S. Bureau of Standards is the determination of the endurance properties of the material. In order to determine which method would be the more searching with regard to the effect of inclusions and other unsound conditions upon the endurance limits, a comparison was made of the endurance limits of a number of ferrous materials determined by both the axial-loading and the rotating-beam methods. Four machines of the usual R. R. Moore type were used for the rotating-beam tests and two Haigh machines were used for the axial-loading tests. Special precautions were taken to secure correct alignment of the latter machines so as to insure truly axial loading.

The materials used were an open-hearth iron; wrought iron in three different conditions as to slag distribution; three plain carbon steels, each in both the annealed and heat-treated conditions; and four rail steels.

The endurance limit determined by the axial-loading method was in no case greater than that determined by the rotating-beam method. The ratios of the endurance limits determined by the axial-loading method to those determined by the rotating-beam method for the various materials are given.

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## Introduction

The determination of the endurance properties of rail steels has formed one phase of an extensive study of this material at the U. S. Bureau of Standards. In planning these studies the question arose as to the most suitable and discriminating method to use for determining the endurance limit of a material such as rail steel which often is far from being "clean" and which may also be unsound for other reasons. The point to be decided was whether the tests should be made by the use of the rotating-beam method or by the axial-loading method. In endurance testing, when a material is loaded as a beam which is constantly rotated, only a relatively thin surface layer is stressed to the desired maximum and it would appear that any in-

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[1] Published by permission of the Director of the Bureau of Standards of the U. S. Department of Commerce, Washington, D. C.

[2] Assistant Metallurgist, U. S. Bureau of Standards, Washington, D. C.

homogeneities in the metal which might affect the endurance properties, such as inclusions or "shatter cracks" located in the interior of the specimen might exert little influence. On the other hand, under conditions of axial loading, the specimen is uniformly stressed throughout its entire cross-section. Consequently it would seem that testing under such conditions would be more desirable for such a material as rail steel in that the test would be a more searching one for showing the effect of inhomogeneties which are ordinarily expected to affect adversely the endurance properties.

There is, however, an anomalous situation with respect to endurance testing by these two methods. As a result of some of the early tests carried out at the University of Illinois, Moore and Jasper[1] concluded tentatively, on the basis of results obtained with 13 lots of iron and steel ranging from 0.02 to 1.20 per cent carbon, that the endurance limit of steel in reversed axial stress should not be regarded as more than 60 per cent of that obtained in reversed flexural stress (rotating beam). Somewhat later, Irwin reported before this Society[2] results he had obtained on high and medium-carbon steel, two nickel-chromium steels and manganese bronze in a Haigh axial-loading machine. The aligning of the specimens in the machine was carried out with great care and the results indicated that the ratio of the endurance limit obtained by axial loading to that by flexural stress varied from 0.95 to 1.10. This work indicated that the lower limits reported previously by Moore and Jasper[1] for axial loading were to be attributed to deviations from true alignment of the specimen. In a later report, Moore and Jasper[3] gave this explanation as probable.

Somewhat later Gough and Tapsell[4] published results of tests carried out during the years 1922 to 1925, that is, before the publication of Irwin's work, which showed that the endurance limit obtained in axial loading is lower than that obtained by the rotating-beam method. In their initial work, in four cases out of seven, the endurance limits obtained by both methods were practically identical, but in three cases the endurance limit obtained by axial loading was lower than that obtained by the other method. Series of runs then were made on two steels (0.13 and 0.33 per cent carbon) in three different Haigh axial-loading machines. The results from the three machines agreed very well among themselves and an endurance limit was reported for the low-carbon steel which was 18 per cent lower than that obtained by the rotating-beam method. For the steel

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[1] H. F. Moore and T. M. Jasper, "An Investigation of the Fatigue of Metals," Series of 1923. *Bulletin 142*, University of Illinois Engineering Experiment Station, Vol. 21, p. 7 (1924).

[2] P. L. Irwin, "Fatigue of Metals by Direct Stress," *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part II, p. 53 (1925).

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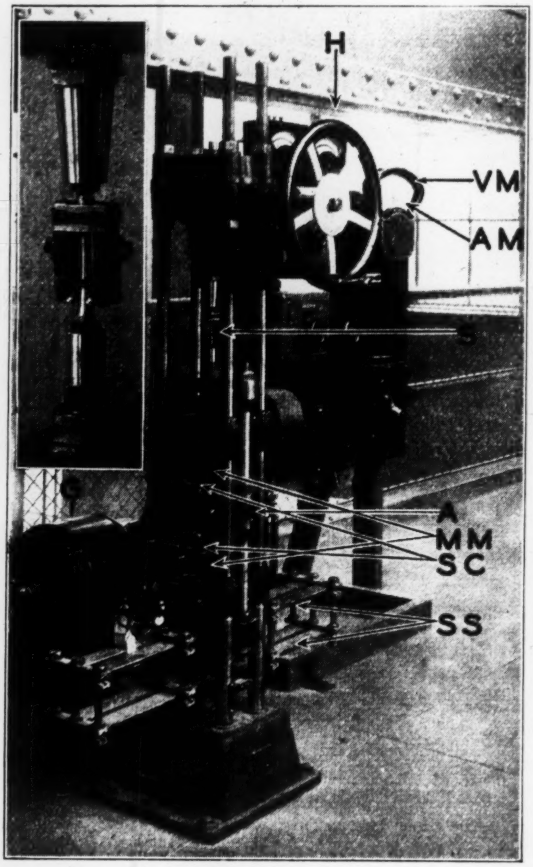

FIG. 1.—View of Haigh Alternating-Stress Testing Machine With Insert Showing Specimen in Grips with Tuckerman Extensometers Attached.

of 0.33 per cent carbon, the endurance limit in axial loading was reported as 26 per cent below the corresponding value obtained by the rotating-beam method.

This unsatisfactory situation was realized in undertaking the work reported upon in this paper and the tests were planned accordingly.

## APPARATUS

All the tests for determining the endurance limit when the specimen was loaded as a rotating beam were made on four machines designed and described by R. R. Moore[1] and commonly referred to by his name. A distinguishing feature of this machine is the relatively short specimen used as compared with other machines of this general type. This machine and its operation are so familiar to testing engineers that a detailed description is considered unnecessary.

The tests for determining the endurance limit under conditions of alternating axial tensile and compressive stresses were made on two[2] machines of the same kind as used by Irwin.[3] This type is usually referred to as the Haigh alternating stress testing machine[4] and its use for routine testing is rare in this country. A short description of the operation of the machine is given here. Figure 1 is a view of the machine, with a close-up view of the specimen with extensometers attached shown in the insert.

The test specimen, *S*, threaded at both ends, is rigidly held at its upper end in a head whose vertical position can be adjusted by means of a hand wheel, *H*. The lower end is connected to a frame to which is bolted an armature, *A*, and the springs, *SS*. The armature, which is held centrally by means of guide springs, oscillates between the poles of two electromagnets, *MM*, excited from a two-phase alternator, *G*, specially wound to give a sine-wave of e.m.f. The alternator is driven at such speed as to give a frequency of vibration to the armature of 2400 cycles per minute.

A differential ammeter, *AM*, indicates by a zero reading equality of current to the two magnet circuits, which is necessary for equal pull in each direction.

A measure of the force applied to the test specimen is obtained by means of a special instrument or "stress meter," *VM*, connected to four secondary coils, *SC*, which surround the poles of the magnets. The stress meter readings are calibrated in terms of the stress-strain relationship determined on a steel calibration bar of similar size and shape as the test specimens. The stress-strain relationship of the calibration bar is determined by mounting the bar in a tension testing machine and measuring, by means of a Tuckerman optical strain gage,[1] the extension of the bar under various applied loads, all of which are within the elastic range of the material. The calibration bar with extensometers is then transferred to the Haigh machine and the relationship is determined between stress-meter readings of the machine and strain as indicated by the extensometer. From the load required to obtain a given strain in the tension machine, it is a simple matter to determine the load on the specimen in the Haigh machine corresponding to a given stress-meter reading.

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[1] R. R. Moore, "Some Fatigue Tests on Non-Ferrous Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part II, p. 66 (1925).

[2] One of these machines was lent to this Bureau by the Watertown Arsenal, Ordnance Department, U. S. Army, for use in this investigation, and grateful acknowledgment is made for this cooperation.

[3] P. L. Irwin, "Fatigue of Metals by Direct Stress," *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part II, p. 53 (1925).

[4] B. P. Haigh, "Experiments on the Fatigue of Brasses," *Journal*, Inst. Metals, Vol. 18, No. 2, p. 55 (1917).

It is recognized that the stress-strain relationship of steel under static tensile stress may be different from that under conditions of repeated stress, as in the Haigh machine, particularly in view of the fact that in the Haigh machine the specimen or calibration bar is subjected to a stress cycle of equal tension and compression. However, no evidence or data are available that indicate the difference to be appreciable for the cycle frequency used (2400 per minute), and it is believed to be negligible. The load that can be applied to the specimen in either tension or compression is limited to the range 500 to 1500 lb. and is varied by varying the current to the field of the alternator which excites the magnets.

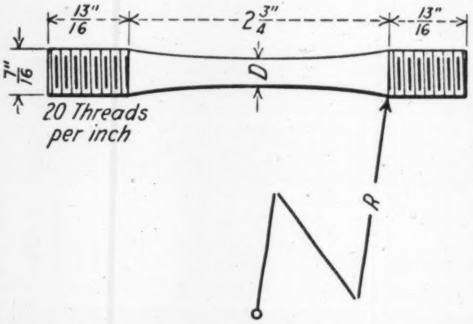

FIG. 2.—Design of Specimen for Haigh Machine.

Diameter $D$ varies according to strength of material from 0.200 to 0.250 in. Radius $R$ varies according to diameter from $9\frac{7}{8}$ to $9\frac{15}{16}$ in.

Since the discrepancies mentioned previously in the results obtained by several investigators in their work with axial-loading endurance testing were believed to have been due to imperfect alignment so that truly axial loading was not obtained in some cases, special pains were taken to align carefully the machines used in the present investigation to avoid deviations from truly axial loading.

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[1] L. B. Tuckerman, "Optical Strain Gages and Extensometers," *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part II, p. 602 (1923).

In order to be able to obtain specimens of the rail steels from various portions of the head of the rail, it was decided to use test specimens of the design shown in Fig. 2. All machining, including cutting of the threads, was done between the same centers on a lathe and the threads were thus coaxial with the rest of the specimen within the precision of the lathe. The threads were turned to fit a thread gage within very close tolerance.

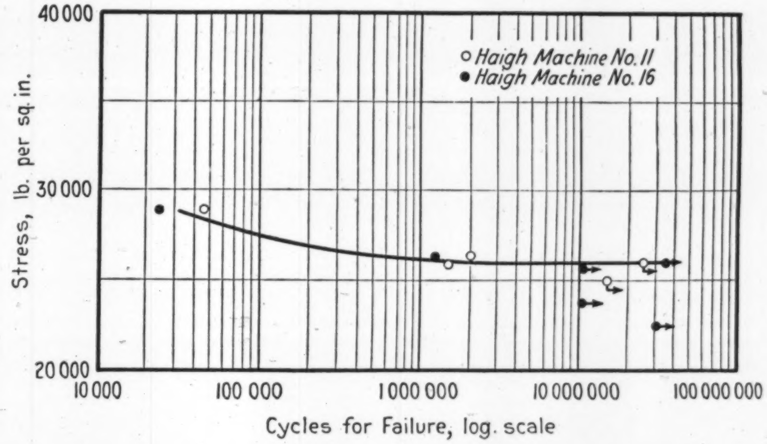

FIG. 3.—Endurance Curve for Open-Hearth Iron, 0.02 per cent Carbon, Obtained on the Two Haigh Machines.

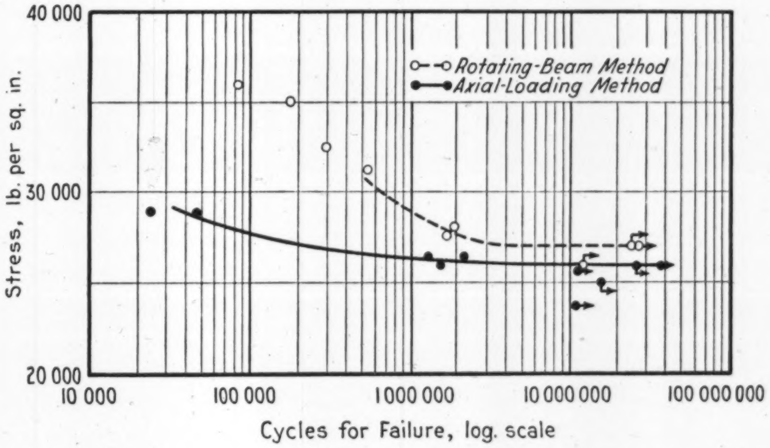

FIG. 4.—Endurance Curves for Open-Hearth Iron, 0.02 per cent Carbon, Obtained by the Two Methods of Test.

Both machines were carefully overhauled and every possible precaution was taken in reassembling them to obtain as perfect align-

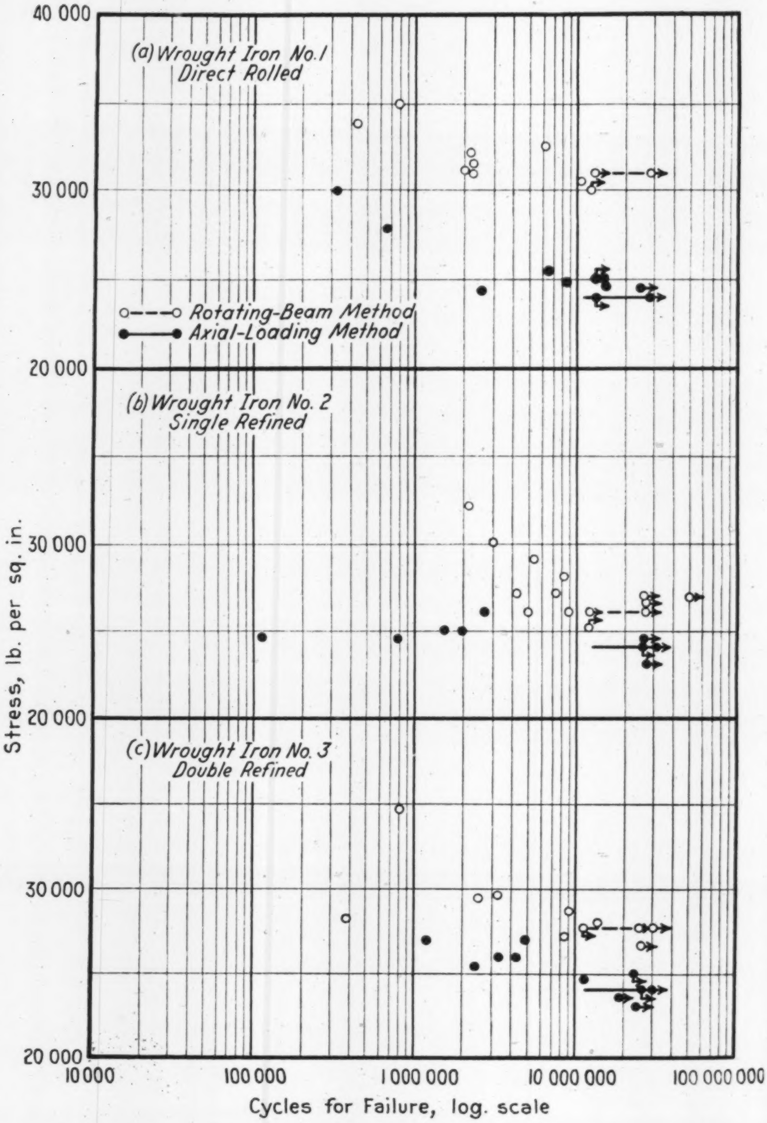

FIG. 5.—Endurance Curves for Wrought Iron Obtained by the Two Methods of Test.

ment as possible. An alignment bar about 8 in. in length and ground true over its entire length was made to fit with a "cushion fit" into the bores of two special adapters which were in turn fastened to the upper and lower heads of the machines in place of the test-specimen grips originally on the machines. The axis of the upper head was then adjusted with respect to that of the lower so that the alignment bar moved freely through both adapters without any binding. The bottom adapter was left in position and formed part of the fixture for holding the lower end of the test specimens. The upper adapter was replaced with a similar one having an internal thread to take the upper end of the specimens.

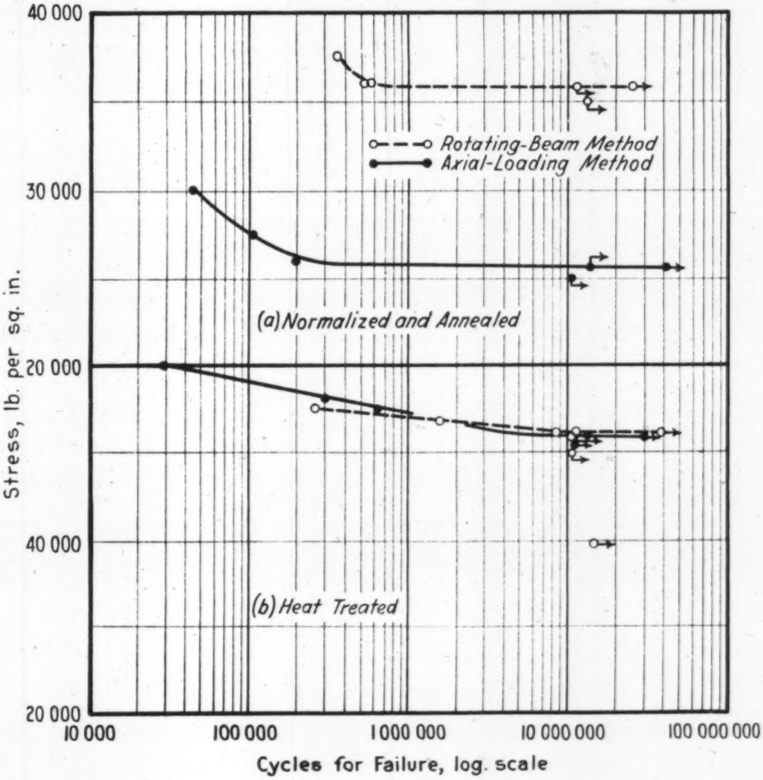

FIG. 6.—Endurance Curves for 0.45-per-cent Carbon Steel Obtained by the Two Methods of Test.

The "feel" of the specimen when turned into the adapters serves as a good check on the alignment of the heads of the machine with the specimen, for it is obvious that a specimen would not run easily in the threads simultaneously in both adapters if there were appreciable lack of alignment.

Before proceeding with a test, a pair of Tuckerman strain gages were attached to opposite sides of the specimen and the deflections under the applied load were measured. Another set of readings was taken with the gages turned through 90 deg. from their original position. In no case have the differences between the readings of the two gages nor of the gages in the two positions been greater than the experimental error in reading the gages.

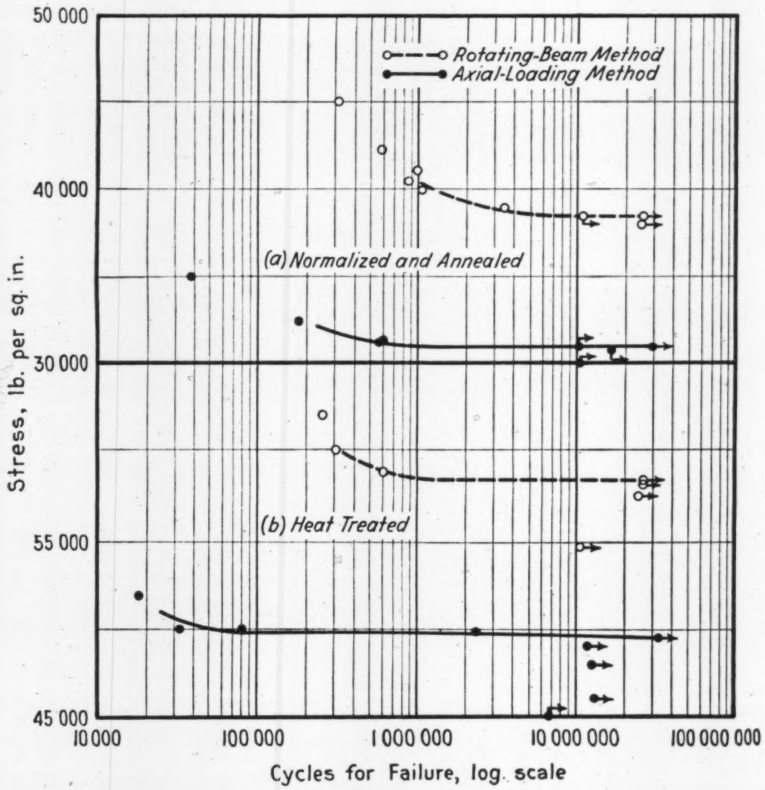

FIG. 7.—Endurance Curves for 0.47-per-cent Carbon Steel Obtained by the Two Methods of Test.

As a verification of the fact that truly axial loading was obtained in the tests here reported, a number of specimens that had failed by partial fracture through the cross-section were removed from the machine and rolled on a piece of plate glass. In all cases they rolled true, which could hardly have been so if bending stresses had been present when the specimens cracked.[1]

## MATERIALS

In addition to rail steels, a number of other materials were tested by the two methods. The following materials are reported upon:

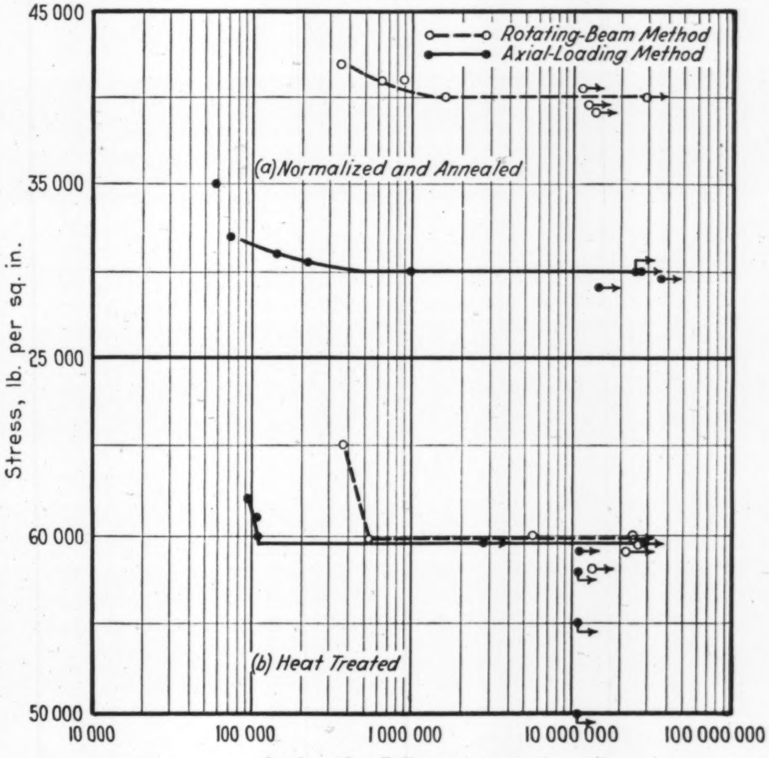

FIG. 8.—Endurance Curves for 0.87-per-cent Carbon Steel Obtained by the Two Methods of Test.

open-hearth iron; wrought iron; plain carbon steels with carbon contents of 0.45, 0.47, and 0.87 per cent, respectively; and four rail steels.

The open-hearth iron was commercial hot-rolled material in 1-in. rounds. The wrought iron, though made commercially, was prepared especially for investigative purposes[2] and should not be classed as commercial material. This was in the form of 1-in. rounds. The three wrought irons differed in the amount of mechanical working received; hence their structural condition with respect to the size and distribution of the slag threads differed also. The condition of the three is indicated by the descriptive terms, direct rolled, single refined and double refined.

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[1] The procedures used for calibration and alignment of the Haigh machines were developed under the direct supervision of Mr. J. R. Freeman, Jr., formerly Chief of the Mechanical Metallurgy Section of the Bureau of Standards.

[2] H. S. Rawdon and O. A. Knight, "Comparative Properties of Wrought Iron Made by Hand Puddling and by the Aston Process," Research Paper 124, *Bureau of Standards Journal of Research*, Vol. 3, p. 952 (1929).

The steels reported upon were in the form of flat bars, $2\frac{1}{4}$ by $\frac{3}{4}$ in. except the steel of 0.45 per cent carbon content which was in the form of $\frac{3}{4}$-in. rounds. The condition with respect to heat treatment of the three steels is given later in the paper in Table I which summarizes the results of the tests. The composition of the four rail steels was as follows:

| Designation of Rail | Composition, per cent | | | | | Remarks |
|---|---|---|---|---|---|---|
| | Carbon | Manganese | Phosphorus | Sulfur | Silicon | |
| CN2............ | 0.75 | 0.73 | 0.030 | 0.030 | 0.17 | Used rail, companion rail developed a transverse fissure. |
| M1D............ | 0.66 | 1.53 | 0.030 | 0.057 | 0.29 | New rail. |
| H3F............ | 0.82 | 0.67 | 0.023 | 0.040 | 0.34 | New rail. |
| L15............ | 0.70 | .... | ..... | ..... | .... | Used rail, rolled from hot-top ingot. |

Specimens were taken in groups of six from the head of each rail. Each group consisted of two rows of three specimens arranged symmetrically with respect to the vertical medial section of the rail and also the horizontal medial section of the head.[1]

## RESULTS

In Table I is given a summary of the results obtained with the various materials used. The conventional *S-N* diagrams summarizing graphically the results for each particular material are also given in Figs. 3 to 12. These should be consulted particularly for the inhomogeneous materials, such as wrought iron, which would be expected to show considerable scatter in the test results.

The results given in Fig. 3 illustrate the agreement obtained with the two Haigh machines when used to test the same material.

The endurance limits (rotating-beam method) for the rail steels had been determined in the course of two previous investigations.[2] Only the axial-loading tests were made during the present investigation.

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[1] This method of sampling has been used in previous papers on rails. See Bureau of Standards *Technologic Paper No. 363*, p. 275.

[2] The results for the rotating-beam method for rails L15 and CN2 are from U. S. Bureau of Standards *Research Paper 92*, pp 222 and 239, respectively; those for rails M1D and H3F from U. S. Bureau of Standards *Research Paper 182*, pp. 859 and 867, respectively.

## DISCUSSION

The results obtained for open-hearth iron indicate that for a uniform ductile material of this kind the difference in the endurance limits, as determined by the two methods, is small and probably of little significance. It may be pointed out, however, that the endurance limit as determined by axial loading was slightly below that determined by the comparison method.

The results obtained with wrought iron, Fig. 5, show considerable scatter as might be expected in such an inhomogeneous material. The three wrought irons used were identical except for the amount of refining, that is, piling and rolling, they had received. The fact that the endurance limit for each of the three grades as determined under conditions of axial loading was practically identical is believed to be of real significance, and to depend on the fact that the total amount of included slag was essentially the same in each. It will be noted that in the case of the two wrought irons, Nos. 2 and 3, which had received more refining than No. 1, the endurance limit as determined by the rotating-beam method was considerably lower than that of the direct rolled No. 1 iron. This may properly be attributed to the more uniform distribution of the slag threads throughout the material so that their effect was more readily shown by the rotating-beam method.

TABLE I.—RESULTS OF ENDURANCE TESTS OF STEEL.

| Material | Carbon, per cent | Heat Treatment, deg. Fahr. | | | | Rockwell Hardness, "B" Scale | Endurance Limit, lb. per sq. in. | | Endurance Ratio[c] |
|---|---|---|---|---|---|---|---|---|---|
| | | Normalized | Annealed | Quenched[b] | Tempered | | Rotating-Beam Method | Axial-Loading Method | |
| Open-hearth Iron[a] | 0.02 | .... | .... | .... | .... | 47 | 27 000 | 26 000 | 0.96 |
| Wrought Iron No. 1[a] | .... | .... | .... | .... | .... | .. | 31 000 | 24 000 | 0.77 |
| Wrought Iron No. 2[a] | .... | .... | .... | .... | .... | .. | 26 500 | 24 000 | 0.91 |
| Wrought Iron No. 3[a] | .... | .... | .... | .... | .... | .. | 28 000 | 24 000 | 0.86 |
| Plain Carbon Steel | 0.45 | 1610 | 1470 | .... | .... | 73 | 35 800 | 26 500 | 0.74 |
| Plain Carbon Steel | 0.45 | 1610 | 1470 | 1525 | 1105 | 84 | 46 200 | 45 800 | 0.992 |
| Plain Carbon Steel | 0.47 | 1615 | 1475 | .... | .... | 80 | 38 500 | 31 000 | 0.80 |
| Plain Carbon Steel | 0.47 | 1615 | 1475 | 1525 | 1000 | 89 | 58 400 | 49 500 | 0.85 |
| Plain Carbon Steel | 0.87 | 1490 | 1400 | .... | .... | 86 | 40 000 | 30 000 | 0.75 |
| Plain Carbon Steel | 0.87 | 1490 | 1400 | 1485 | 1000 | 100 | 59 800 | 59 600 | 0.996 |
| Rail Steel CN2 | 0.75 | .... | .... | .... | .... | 102 | 44 200 | 41 500 | 0.94 |
| Rail Steel M1D | 0.66 | .... | .... | .... | .... | 97 | 69 600 | 57 500 | 0.82 |
| Rail Steel H3F | 0.82 | .... | .... | .... | .... | 102 | 55 800 | 47 000 | 0.82 |
| Rail Steel L15 | 0.70 | .... | .... | .... | .... | 99 | 55 000 | 44 500 | 0.81 |

[a] Tested in rolled condition.
[b] All material quenched in oil from temperature indicated.
[c] Ratio of axial-loading method to rotating-beam method.

The results given for the four carbon steels in the annealed state are representative of several others ranging in carbon content from 0.28 to 0.93 per cent. In every case, the endurance limit determined under conditions of axial stress was below that obtained by repeated flexural stress. The results obtained with the medium-carbon steel (0.45 per cent carbon) and the eutectoid steel (0.87 per cent carbon)

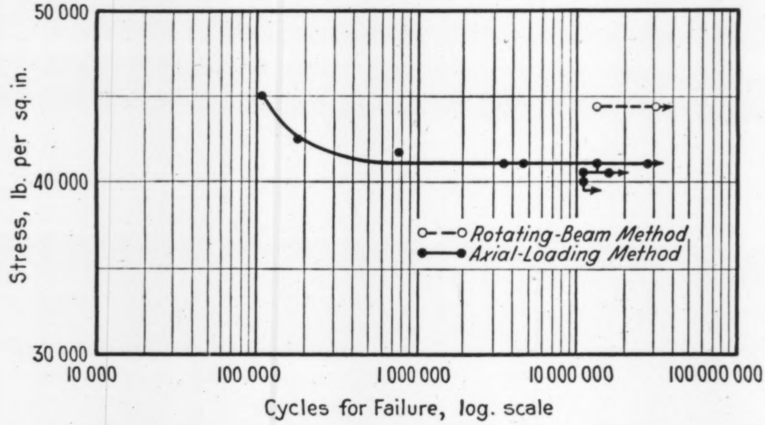


FIG. 9.—Endurance Curve for Rail Steel from Used Rail (CN2) Determined by the Axial-Loading Method Compared with the Endurance Limit Determined by the Rotating-Beam Method.

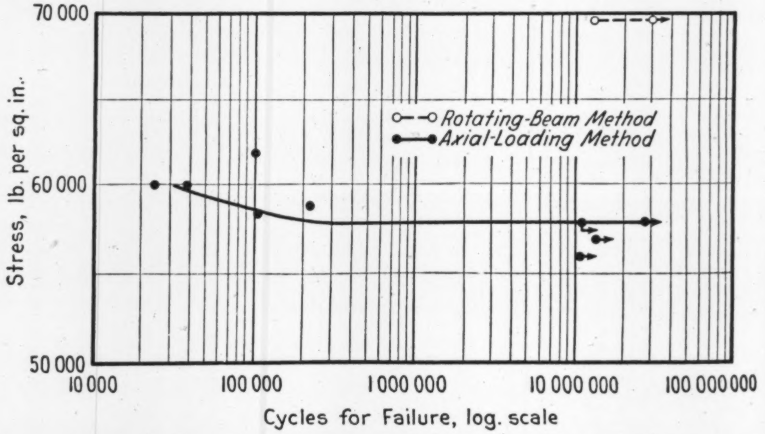


FIG. 10.—Endurance Curve for Medium Manganese Rail Steel from New Rail (M1D) Determined by the Axial-Loading Method Compared with the Endurance Limit Determined by Rotating-Beam Method.

after heat treatment are, however, of greatest significance. The effect of heat treatment in raising the endurance limit was, of course, to be expected. The change in the ratio of the endurance limits as determined by the two methods was totally unexpected, however.

As shown in Figs. 6 (*b*) and 8 (*b*), the two methods of testing gave identical results in each case for the two steels after heat treatment, whereas the endurance limits as determined by the two methods on the same materials before heat treatment, Figs. 6 (*a*) and 8 (*a*), differed widely, the value obtained by axial stressing being the lower in each case. That the effect of heat treatment cannot always be predicted is illustrated by the results obtained with the steel of 0.47 per cent carbon content. As shown in Fig. 7 the endurance limits

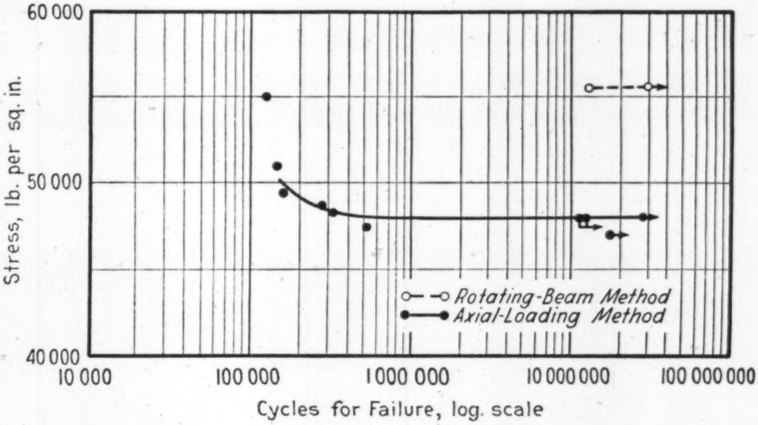


FIG. 11.—Endurance Curve for "Normal Practice" Rail Steel (Rail H3F) Determined by the Axial-Loading Method Compared with the Endurance Limit Determined by the Rotating-Beam Method.

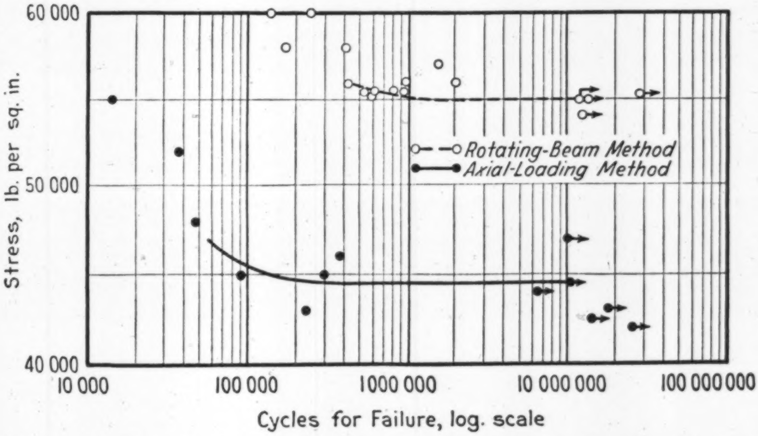
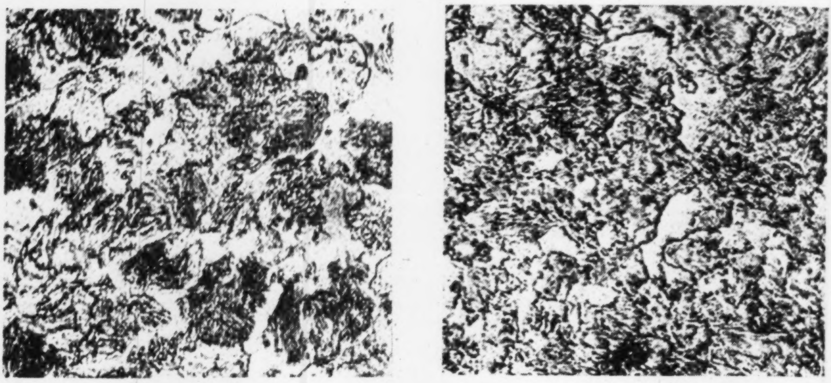
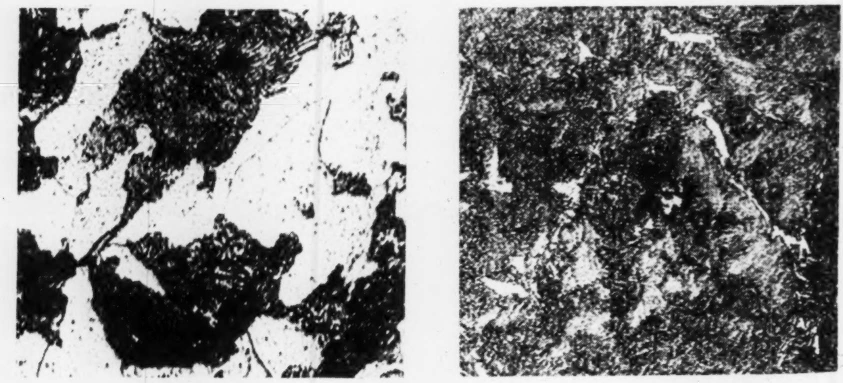


FIG. 12.—Endurance Curves for Used Rail Steel From Hot-Top Ingot (Rail L15) Obtained by the Two Methods of Test.



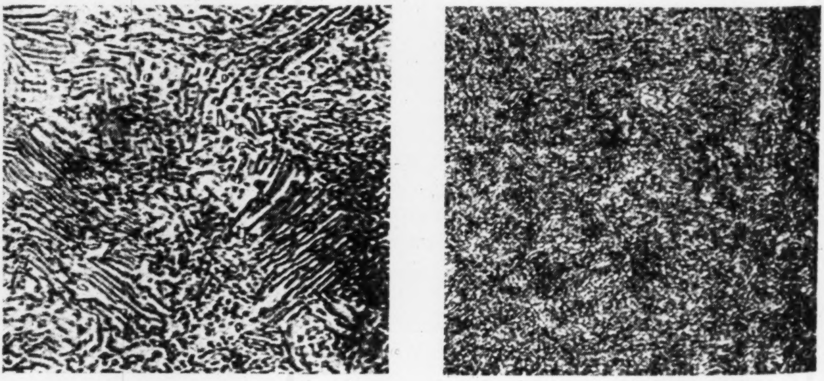
(*a*) 0.45-per-cent Carbon Steel, Normalized at 1610° F., Annealed at 1470° F.

(*b*) 0.45-per-cent Carbon Steel, Normalized at 1610° F., Annealed at 1470° F., Quenched in Oil from 1525° F., Tempered at 1105° F.



(*c*) 0.47-per-cent Carbon Steel, Normalized at 1615° F., Annealed at 1475° F.

(*d*) 0.47-per-cent Carbon Steel, Normalized at 1615° F., Annealed at 1475° F., Quenched in Oil from 1525° F., Tempered at 1000° F.



(*e*) 0.87-per-cent Carbon Steel, Normalized at 1490° F., Annealed at 1400° F.

(*f*) 0.87-per-cent Carbon Steel, Normalized at 1490° F., Annealed at 1400° F., Quenched in Oil from 1485° F., Tempered at 1000° F.

FIG. 13.—Structures of the Three Carbon Steels Used in the Comparison of the Two Methods of Test (× 500). Etchant, 2 per cent Nital.

determined for this material by the two methods differed considerably after heat treatment but not so widely as for the annealed steel. A completely satisfactory explanation for this difference in behavior has not yet been found. The examination of the microstructure throws some light, however, as shown in Fig. 13. It will be seen that the 0.45 per cent carbon and the 0.87 per cent carbon heat-treated steels were the more uniform in their structure and consisted of much smaller grains. The structure of the 0.47 per cent carbon steel was coarser and remnants of the preexisting ferrite envelopes still persisted. Evidently there is need for more study on this point.

The results obtained for the four rail steels, Figs. 9, 10, 11 and 12, were consistent with those obtained with the other materials in that the endurance limit for conditions of axial stress was below that obtained by the rotating-beam method.

It is believed that the foregoing results should be interpreted as showing that the endurance limit of iron or steel as determined under conditions of axial stress may often be very considerably lower than the endurance limit of the same materials when determined under conditions of repeated flexural stress. The explanation previously advanced by some experimenters that the relatively lower endurance limits obtained by the method of axial stress should be attributed to the lack of true alignment of the specimens in the machine will not account for many of the differences obtained in this present work. In support of this conclusion are two outstanding cases: (*a*) the results obtained with wrought iron in which the size and distribution of the slag threads were varied, the total slag content remaining unchanged, and (*b*) the marked change which was produced in the ratio of the endurance limits by the two methods of test in the same steel by heat treating the steel.

*Acknowledgment:*

The author gratefully acknowledges his indebtedness to the following former members of the Metallurgical Division of the Bureau of Standards: H. W. Gillett, former Chief of the Division of Metallurgy who initiated this work; J. R. Freeman, Jr., former Chief, Section of Mechanical Metallurgy who planned the investigation, carried out the calibration of the machines and directed a great deal of the early experimental work; and H. N. Solakian who carried out many of the tests in the preliminary stages of the investigation. To W. H. Swanger, Chief, Section of Mechanical Metallurgy under whose direction the work has been carried to completion, the author is also greatly indebted.

# DISCUSSION

MR. J. R. FREEMAN, JR.[1]—There is one point that I think is worthy of emphasis in the work described by Mr. France and that is the fact that the tests were duplicated on two separate machines. In the earlier days of the work we were in contact with Mr. Irwin of the Westinghouse Electric and Manufacturing Co. with the hope that we might interchange calibrations of their machine and our machine. We had only one at that time. Unfortunately our calibration equipment could not be readily adopted to their machine. We obtained the second machine later and I should like to emphasize again the fact that the two machines giving the same results on the same material is a very definite confirmation of the validity of the results. The probability of any misalignment that might be present being identical in each machine is extremely small.

The work on rail steels is of importance, and I should like to call attention to the rather marked difference that is apparent in the fatigue properties of rail steels as indicated in the curves when tested under the conditions of the rotating beam and the conditions of axial loading. In all cases under axial loading the fatigue strength is less. We know from extensive tests made at the Bureau of rail steels under the conditions of rotating beams[2] and stress analyses that have been made in particular by H. F. Moore that rails in service may be operating in many cases quite close to their fatigue limits. The fact that under the conditions of reversed axial stress the fatigue limit is lower than indicated by the rotating-beam tests narrows the margin between service stress and fatigue limits and should be given serious consideration.

MR. G. S. VON HEYDEKAMPF.[3]—Another comparison in bending and axial fatigue loading has been made in Germany by Lehr (Thesis, Stuttgart, 1925). He used rotating-beam machines of a type similar to the R. R. Moore machine. The axially loading machine is similar to the Haigh machine. The frequency of Lehr's machine was, however, much higher, namely, 30,000 per minute. His comparison tests are consequently not directly comparable with those presented by the author. Lehr obtained fatigue limits 10 to 15 per cent higher in axial than in flexural loading. It is not clear whether this discrepancy is due to the higher frequency applied, or to any unknown disturbances.

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[1] Technical Department, American Brass Co., Waterbury, Conn.

[2] J. R. Freeman, Jr., and R. D. France, "Endurance Properties of Some Special Rail Steels," *Bureau of Standards Journal of Research*, Vol. 4, June, 1930.

J. R. Freeman, Jr., and H. N. Solakian, "Effect of Service on Endurance Properties of Rail Steels," *Bureau of Standards Journal of Research*, Vol. 3, August, 1929.

J. R. Freeman, Jr., R. L. Dowdell and W. J. Berry, "Endurance and Other Properties of Rail Steels," U. S. Bureau of Standards *Technologic Paper No. 363*.

[3] Research Engineer, Baldwin-Southwark Corp., Southwark Foundry and Machine Co., Division, Philadelphia, Pa.

The author reported that, as a check of good alignment, the axially loaded specimens were rolled on a glass plane and found to run truly, even when a fatigue failure had destroyed part of the cross-section. The same method has been in use with rotating-beam specimens (1 in. in diameter, 2½ ft. long), at the Woehler Institute. These specimens were removed from the machine as soon as slight deviations from the theoretical deflection indicated a starting fatigue failure.

Although bending stresses represent an extreme case of misalignment, these partly cracked flexural specimens were still exactly straight. From this experience, no indications of misalignment in the case of axially stressed specimens should be expected, when using the method mentioned.

**THE CHAIRMAN** (*Mr. H. W. Gillett*[1]).—Of course one has to take those very high frequency figures with a grain of salt before you draw conclusions as to the effect of high frequency or axial loading. The English experiments on very high frequency certainly need a lot of corroboration before we can take them at their face value. It is difficult enough to measure stresses on the Haigh machine or anything else with real accuracy, but when you get up to super-speeds you ganerally do not know what you have, and it is a pretty safe bet you have not what you think you have. We have to be extremely careful in drawing any conclusion. It seems to me that the author has very carefully checked the stress calculations and everything else. Moreover it certainly appeals to reason, one should expect in non-uniform material a lower fatigue value on axial loading than on a rotating-beam test. It seems to me that that question comes pretty nearly to being solved, and I think this is a paper which will be one of the landmarks in fatigue testing.

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[1] Director, Battelle Memorial Inst., Columbus, Ohio.

# FATIGUE TESTS OF WELD METAL

By R. E. Peterson[1] and C. H. Jennings[2]

## Synopsis

In this paper, fatigue test data are given on weld metal proper, covering effects of machining, annealing and peening. A particular feature of this investigation is the development of a rough (not machined) weld metal fatigue test specimen. Machining is shown to reduce the fatigue strength, strain annealing to have no large effect, and peening to increase the fatigue strength.

## Introduction

In the analysis of welded joints subjected to repeated stresses, it has been possible in many cases to solve the stress distribution mathematically but yet not be able to predict the path of failure or the strength of the joint, because information concerning the endurance properties of weld metal proper (particularly in the not-machined state) has not been available. It is quite out of the question to test every design of joint proposed for dynamically stressed weld-fabricated structures,[3, 4] but rather it is hoped that by accumulation of data along the lines indicated in this paper the analysis of proposed designs for dynamic service can be performed logically in all cases.

The effects of machining, annealing and peening have been investigated independently in the tests herein described. It is felt that even though the absolute values of endurance limits given in this paper may be changed somewhat by use of a different welding current, electrode, etc., the effects mentioned above will quite likely be in the same directions. Further tests, particularly with respect to size effect, are now in progress.

## Test Details

Rotating-cantilever-beam fatigue testing equipment (large size) described previously[5] was used for this investigation. The machined weld-metal specimens were made as shown in Fig. 1, being of the usual tapered cantilever-beam design. It is felt that such a specimen of "all weld metal" is essentially different from a butt weld.[1]

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[1] Manager, Mechanics Division, Research Laboratories, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[2] Research Laboratories, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[3] M. Stone and J. G. Ritter, "Electrically Welded Structures Under Dynamic Stress," *Journal*, Am. Inst. Electrical Engrs., March, 1930, p. 202.

[4] A. M. Candy, "Modern Applications of Arc Welding," Symposium on Welding, published by the American Society for Testing Materials (1931).

[5] R. E. Peterson, "Fatigue Tests of Large Specimens," *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part II, p. 371 (1929).

The development of a rough (not-machined) weld metal fatigue test specimen, free from stress-concentration due to form, presented considerable difficulty. The design finally adopted is shown in Fig. 2 (*b*). A test specimen of this type in place in the testing machine is shown in Fig. 3. It will be noticed that the far end of the specimen has fractured. Closer views of the welded portions of test specimens (not-machined) are shown in Fig. 4.

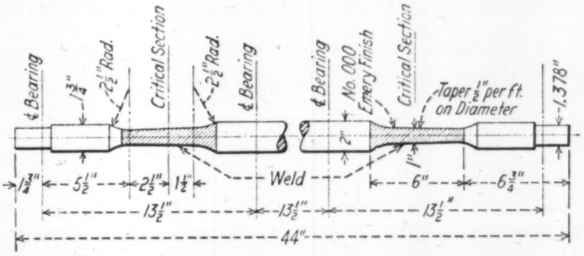
℄ Bearing
1¾"
2½" Rad.
Critical Section
2½" Rad.
℄ Bearing
℄ Bearing
No. 000 Emery Finish
Critical Section
Taper ½" per ft. on Diameter
1.378"
2"
1"
1¾"
5½"
2½"
1½"
Weld
6"
6¾"
13½"
13½"
13½"
44"

FIG. 1.—Machined Weld-Metal Specimen.

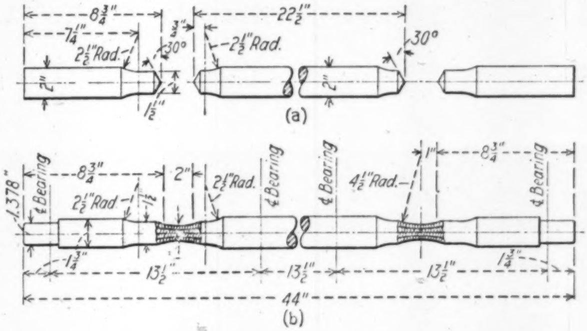
8¾"
7¼"
2½" Rad.
30°
¾"
22½"
2½" Rad.
30°
2"
1½"
2"
(a)
1.378"
℄ Bearing
8¾"
2"
2½" Rad.
1½"
2½" Rad.
℄ Bearing
℄ Bearing
4½" Rad.
1"
8¾"
℄ Bearing
1¾"
13½"
13½"
13½"
1¾"
44"
(b)

FIG. 2.—Showing Fatigue-Test Specimen and Specimen Blank.
(*a*) Rough Weld-Metal Specimen Blanks.
(*b*) Rough Weld-Metal Test Specimen.

The welds were made with a $\frac{5}{32}$-in. diameter, low-carbon Westinghouse Flex Arc Electrode.[2] The blanks shown in Fig. 2 (*a*) were turned from 2-in. diameter hot-rolled, low-carbon steel. The welding current was approximately 150 amperes in all cases.

In welding the rough weld metal specimens, the end blanks, Fig. 2 (*a*), were placed on end and a slender column of weld metal

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[1] G. Lobo, "Strength of Butt Welds," *Journal*, Am. Welding Soc., April, 1929.

[2] This is essentially a bare (uncoated) electrode.

2 in. high built on the point of the machined cone. The three portions that comprise the complete specimen were then lined up in a jig which consisted essentially of an angle placed in a horizontal position, and joined together by welding the built-up columns of weld metal to the center bar. The welds were then built to the desired size by depositing circumferential layers of weld metal, each bead of which was laid in a direction parallel to the axis of the specimen. The contour of the completed weld was controlled by means of a template.

The annealed specimens were held in the furnace at the temperatures indicated (1000 or 1700° F.) for two hours and then allowed to cool slowly to room temperature before removing from the furnace.

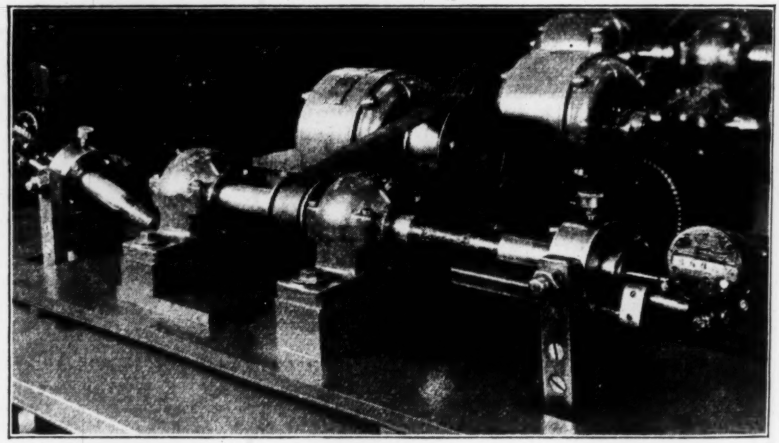

FIG. 3.—Testing Machine with Peened, Rough Weld-Metal Specimen in Place.

The peened specimens were prepared by pounding the rough weld metal surface by using a light air hammer and a blunt ended peening tool. The appearance of a peened surface is seen in Fig. 4. Micrographic examination of the weld metal indicated that the peening affected the structure of the metal to a depth of about 0.01 in. below the surface.

The test data are shown graphically in Figs. 5, 6, and 7. The stresses were computed in the usual manner, that is, by the flexure formula applied to the section of failure.

## DISCUSSION OF RESULTS

Test data obtained from machined specimens (Fig. 1) are shown in Fig. 5, the three curves representing different methods of deposition of weld metal: lengthwise, crosswise, and spiral. The difference between lengthwise and crosswise deposition is small—400 lb. per sq. in., which is within the limits of error of testing weld metal. The spirally deposited weld metal was lower in fatigue strength, but it was noticed that the welds were inferior to the lengthwise and crosswise welds. The spiral welds contained considerable slag and fractured very irregularly. Even so, the total deviation of the machined weld metal endurance limits is but 2300 lb. per sq. in. The direction of deposition does not, therefore, seem to make a great deal of difference in this particular case.

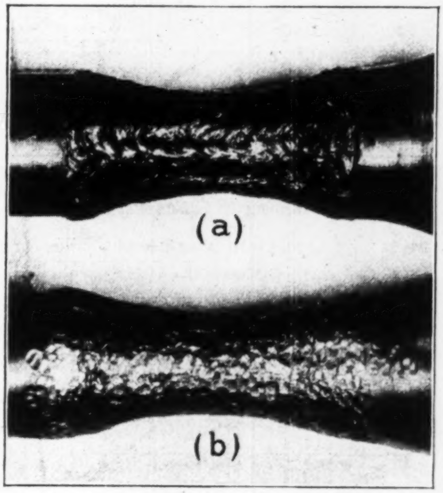

FIG. 4.—Critical Sections of Weld-Metal Specimens.
(*a*) Rough weld metal.
(*b*) Peened weld metal.

If now the results of rough (not-machined) weld metal as shown in Fig. 6 be considered, a surprising difference appears. The rough specimens (see Fig. 4), which were supposedly much weaker in fatigue than the smoothly machined and polished specimens, actually gave higher endurance limits. The same curious phenomenon was also noticed in previous tests on fillet welds,[1] in which case residual stresses were assumed to constitute the major difference.

In order to determine the effect of residual stresses, it was decided to anneal a series of rough weld-metal specimens at 1000° F. for two hours, which procedure has been shown to eliminate most of the residual stresses present in weld metal.[1] The annealed specimens gave an endurance limit 900 lb. per sq. in. higher than those not annealed. This difference is so small that the only conclusion that can be drawn is that strain annealing does not produce a large effect in this particular case.

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[1] R. E. Peterson and C. H. Jennings, "Fatigue Tests of Fillet Welds," *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part II, p. 384 (1930).

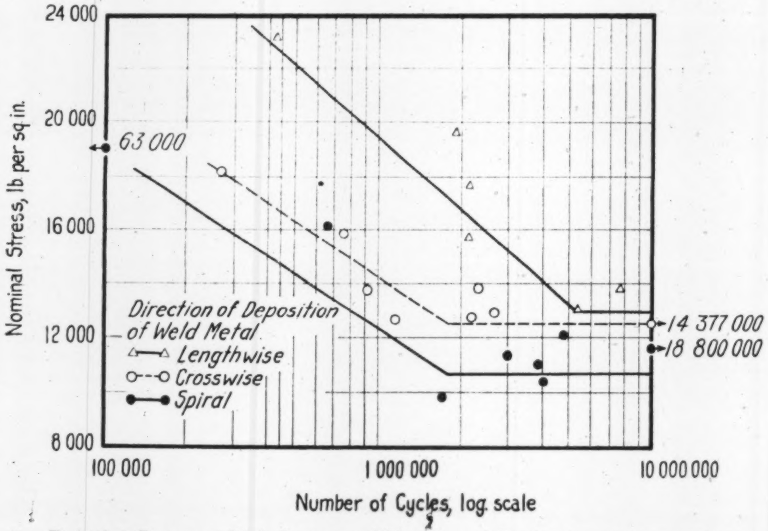

FIG. 5.—Relation Between Unit Stress and Number of Cycles for Failure of Machined Specimens.

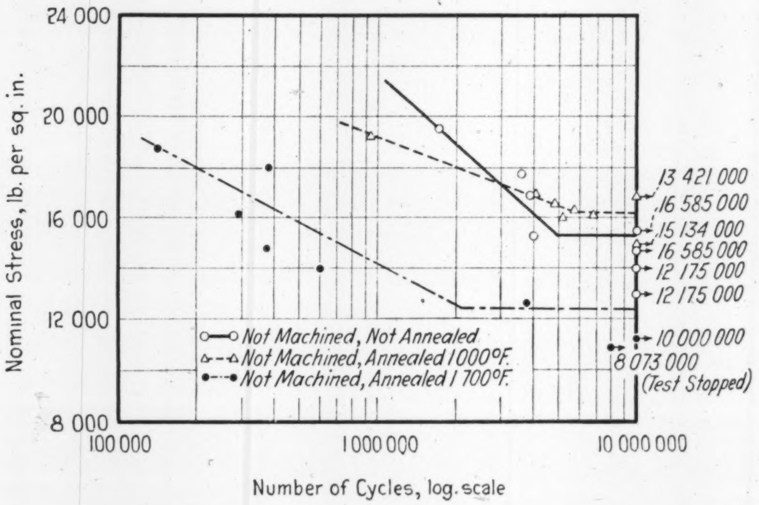

FIG. 6.—Relation Between Unit Stress and Number of Cycles for Failure of Rough Specimens Annealed at Different Temperatures.

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[1] C. H. Jennings, "The Relief of Welding Strains by Annealing," *Journal*, Am. Welding Soc., Vol. 10, No. 9, p. 26, September, 1931.

A series of tests was also made to determine the effect of annealing at 1700° F. Here a substantial decrease of endurance limit was obtained, which seems reasonable since 1700° F. is above the critical point and a coarse grain structure was produced. The strain annealing at 1000° F. produced no change in grain structure.

Since the strain annealing at 1000° F. produced no large difference in endurance strength, it would seem that residual stress is not as important in this particular case as had been thought.[1]

No definite conclusions could be drawn from numerous hardness readings taken on the surfaces of rough and machined weld-metal specimens.

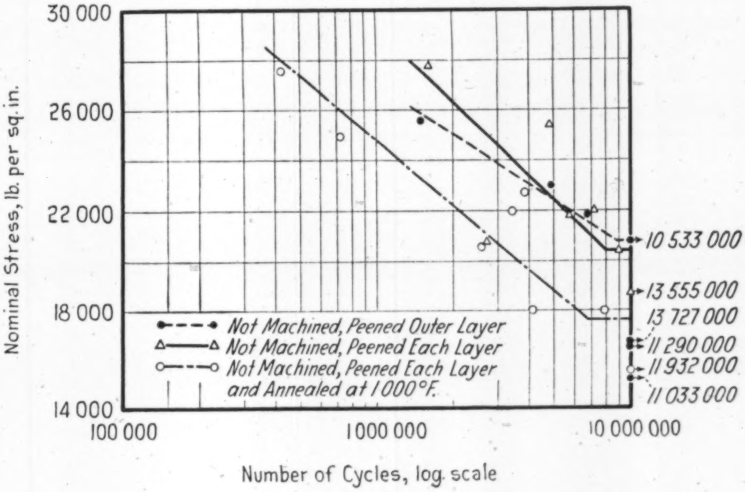

FIG. 7.—Effect of Peening on Relation of Unit Stress and Number of Cycles for Failure.

The only difference apparent to the authors is that the machined weld metal surfaces, if examined carefully, show tiny "pin holes" which are not apparent on the rough skin. This would account for the lowering of endurance strength due to machining. It might be added that the tiny holes seem always to be present in hand-welded metal but are not as prevalent in automatically welded metal.

If the above difference between rough and machined weld metal should be further verified by other investigators, it will have an important bearing on the future testing of welds, since practically all welds are used in the rough state.

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[1] This statement applies to simple joints only and not to composite structures. There is evidence that in some types of composite structures such high residual stresses may be set up as to impair the strength of the structure, in which case annealing is advisable.

The effect of peening is shown in Fig. 4, with test results in Fig. 7. In this particular case, peening raises the endurance limit considerably. Apparently it makes no difference whether each layer is peened successively or only the outer layer is peened. This would be expected since the fatigue crack begins at the outer surface and once started will spread through the cross-section. It has also been pointed out that the effect of peening the inner layers is probably eliminated by the annealing action of subsequent layers. Annealing the completed specimen apparently does not remove all of the effect of peening, since the smoother contour probably has an effect, in addition to the cold working.

The above statements concerning peening are not applicable to fillet welds, since fatigue failure in such a case starts either at the base of the weld or at the inside.[1]

It should be mentioned in conclusion that the values given in this paper should be used rather carefully, since tests on size effect now in progress show higher endurance limits for smaller weld specimens and lower endurance limits for larger weld specimens. It is the intention to publish this work when completed.

## CONCLUSIONS

The following conclusions will be understood to apply to 1-in. diameter weld-metal specimens, hand welded as previously described:

1. The direction of deposition of weld metal does not have a large effect on fatigue strength.
2. Rough (not-machined) weld metal has a higher endurance limit than machined and polished weld metal.
3. Annealing at 1000° F. does not greatly change the fatigue strength of rough weld metal.
4. Annealing at 1700° F. results in a coarser grain structure and a lower fatigue strength.
5. Peening increases the fatigue strength of rough weld metal.
6. Peening the outer layer only is as effective as peening each layer successively.

*Acknowledgment.*—Assistance has been rendered by Mr. F. C. Cassel in the operation of the testing machine. The authors are also indebted to Mr. J. M. Lessells, Manager of Mechanics Division, for his interest and suggestions concerning the investigation.

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[1] R. E. Peterson and C. H. Jennings, "Fatigue Tests of Fillet Welds," ***Proceedings***, Am. Soc. Testing Mats., Vol. 30, Part II, p. 384 (1930).

# DISCUSSION

**Mr. H. E. Smith.**[1]—In actual welding of rather large articles, it has been developed very clearly that in an unannealed weld there is a brittle zone not in the weld metal but in the welded piece just outside the weld. I should like to ask the authors whether the tests described bring out any features in that connection; were any breaks outside the weld? When we do welding in the shop on miscellaneous machine parts or other similar parts of construction it is found to be very necessary to anneal the welds.

**Mr. R. E. Peterson.**[2]—The specimens were designed with a curved portion, so that the neck of the curve is considerably smaller than at the portion where the weld joined the other metal. Consequently, we had no failures at all at the fusion zone. We have noticed, however, that if the specimen is simply a cylindrical test specimen, for instance, using a butt weld, then we are quite apt to have failure in the fusion zone.

**Mr. J. C. Lincoln**[3] (*by letter*).—The specimens which were tested as described by Peterson and Jennings were, according to the paper, welded with a $\frac{5}{32}$-in. diameter low-carbon electrode. This is called the "Westinghouse Flex Arc Electrode."

I understand this electrode is substantially a bare wire, that is, a wire which has only the lime coating on it which is obtained with the ordinary drawing process or this lime coating plus a thin coating which may be obtained by dipping the electrode in a bath and drying the electrode subsequently. In that case the diameter of the steel is substantially equal to the outside diameter of the electrode as used. It should be kept in mind that a substantially bare wire produces a weld metal which is much more brittle than a wire with a coating of such a nature as to control the atmosphere around the arc. Bare wire welds are satisfactory in tensile strength but the elongation is low and if such welds are heated and cooled repeatedly to a temperature of 1500° F., cracking will develop in them. However, if the same electrode is covered with a proper heavy coating of such a nature as to give a reducing atmosphere at the arc, the resulting weld metal will be equal or superior in tensile strength to the weld metal deposited by bare wire and the ductility of the resulting metal will be very much higher than is possible to get with bare wire. The resistance to repeated heating and cooling to a temperature of 1500° F. will be substantially equal to that of the parent metal.

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[1] Engineer of Materials, New York Central Lines, New York City.

[2] Manager, Mechanics Division, Research Laboratories, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[3] Chairman, Lincoln Electric Co., Cleveland, Ohio.

I would expect, therefore, that the fatigue results of bare wire welds might be very different from fatigue results of the same size sample welded with an electrode covering which gives ductile metal instead of brittle metal. A number of other investigators have published results indicating that the resistance to fatigue of welds made with properly covered electrodes is very much higher than can be obtained with bare wire welds and I hope that Messrs. Peterson and Jennings will give us a paper a year from now giving the results of fatigue tests made on the same machine, made in the same way, for both methods of making welds.

MESSRS. R. E. PETERSON[1] AND C. H. JENNINGS[2] (*authors' closure by letter*).—The Westinghouse Flex Arc Electrode is a so-called bare (uncoated) electrode and the results reported should be interpreted as such. The relatively low fatigue strength of manually prepared welds made with uncoated electrodes, although influenced by several factors, is primarily the result of small gas or pinholes which appear to be always present in the body of the weld.

Welds deposited with high-grade fluxed electrodes, and which are composed of metal that is clean, uniform, and free from gas holes, will develop fatigue strengths greater than those obtained from welds made with uncoated electrodes. Recent tests conducted on a Westinghouse developed fluxed electrode by using a beam type specimen 0.273 in. in diameter at the critical section gave an endurance limit of 31,000 lb. per sq. in. This result shows approximately a 40-percent increase in fatigue strength above that obtained with uncoated electrodes by using the same type of specimen. An increase in fatigue strength similar to the above should also be obtained with machined welds when larger test specimens are used.

The effect of an unmachined surface on weld metal deposited with a fluxed electrode cannot be predicted at the present time. If the explanation offered by the authors for the results obtained from uncoated electrodes is correct, however, a slight decrease below the endurance limit of machined weld metal would be expected from the unmachined type of weld.

The fact that fluxed electrodes produce welds with physical properties superior to welds deposited with uncoated electrodes does not mean that they will entirely replace uncoated electrodes. Uncoated electrodes produce welds that are entirely satisfactory for many classes of work[1] and will continue to be used in such cases. Also in certain work such as fillet welds and overhead welding, they afford marked advantages over fluxed electrodes because of the difficulty of properly controlling and floating out the slag produced by the flux. It is because of these facts that the authors have attempted to investigate thoroughly the fatigue properties of welds made with uncoated electrodes. It is planned to present results on welds made with coated electrodes during the coming year.

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[1] Manager, Mechanics Division, Research Laboratories, Westinghouse Electric and Manufacturing Co. East Pittsburgh, Pa.

[2] Research Laboratories, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

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[1] A. M. Candy, "Modern Applications of Arc Welding," Symposium on Welding, published by the American Society for Testing Materials (1931).

# FATIGUE TESTING OF WIRE[1]

By Stephen M. Shelton[2]

## Synopsis

The endurance limits of test coupons with machined surfaces cut from a structural member cannot be relied upon as the true fatigue limit of the structural member itself.

A method is described for the determination of the fatigue limit of wire in its structural shape and condition. The specimens used are long enough to obtain sufficient increase in fiber stress in the medial transverse plane of the specimen without reducing the specimens cross-sectional area. Principles of critical speeds and means of avoiding their effects are outlined briefly.

Results are shown for tests of (1) high-carbon, heat-treated, galvanized steel wire; (2) the same wire with zinc coating removed; and (3) low-carbon steel wire.

The possibility of using the method for fatigue testing of other structural shapes such as pipes and tubes deserves further investigation.

## Introduction

The endurance limit of a metal is arbitrarily defined as a limiting reversible stress, below which the metal will withstand without fracture an indefinitely large number of cycles of stress.[3] The number of stress cycles necessary to cause failure is usually determined in a machine which subjects the metal specimen to cycles of alternate torsion, alternate flexure, or alternate tension and compression.

It is obviously impracticable in most cases to construct machines for testing various structural shapes of metal. Work has been done, however, on large specimens,[4] and there are certain structural shapes which are adaptable for testing in whole or in part.[5, 6, 7, 8]

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[1] Published with the approval of the Director of the Bureau of Standards of the U. S. Department of Commerce, Washington, D. C.

[2] Junior Metallurgist, U. S. Bureau of Standards, Washington, D. C.

[3] Report of Research Committee on Fatigue of Metals, Appendix, *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part I, p. 260 (1930).

[4] R. E. Peterson, "Fatigue Tests of Large Specimens," *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part II, p. 371 (1929).

[5] J. R. Townsend and C. H. Greenall, "Fatigue Studies of Non-Ferrous Sheet Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part II, p. 353 (1929).

[6] C. F. Jenkin and G. D. Lehman, "High Frequency Fatigue," *Proceedings*, Royal Soc. of London, A, Vol. 125, No. 796, August 1, 1929, p. 83.

[7] H. F. Moore, S. W. Lyon and W. J. A. Cleman, "Tests of the Fatigue Strength of Steam Turbine Blade Shapes," *Bulletin No. 183*, University of Illinois.

[8] J. Bradley, "A High-Speed Endurance Testing Machine for Leaf Springs," *Engineering* (London), January 11, 1929, p. 36.

The usual practice has been to cut test coupons from a structural member and machine them to a convenient size and shape. Any method of attachment of the specimen in the machine is a source of indeterminate local stress concentration. For this reason it is desirable to design the specimen so that the point of maximum stress will be located at some distance from the ends or near the medial transverse plane. This is brought about by a gradual reduction in cross-sectional area from the ends to a minimum at the center. In order to reduce surface effects to a uniform minimum, the surface is finished by grinding to an arbitrary fineness. The specimen is then subjected to predetermined cycles of stress under these controlled conditions until failure occurs.

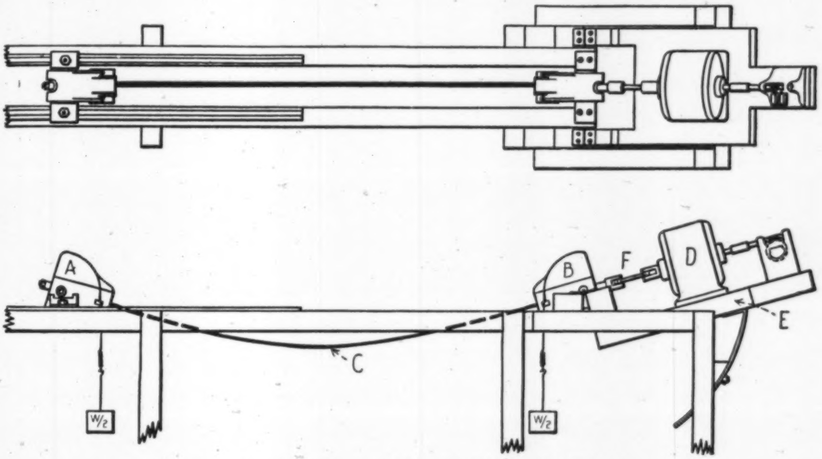

FIG. 1.—Diagram of Wire Fatigue Machine.

Such a procedure can only approximate the true fatigue limit of the metal as it exists in its structural shape. The aggregate effect of change in texture or composition within the metal and of any irregularities at the surface caused in the process of fabrication can only be surmised. It is well known that these factors have a major effect on the fatigue limit of a test bar.[1] Specimens whose surfaces were arbitrarily deformed, notched, corroded or alloyed with other metals have been tested in comparison with specimens of like material whose surfaces were polished in the usual manner. Results show that the effects of surface conditions vary with different materials and different methods of fatigue testing. It is apparent that methods by which metal can be tested in the condition in which it is used in service are of value.

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[1] Report of Research Committee on Fatigue of Metals, Appendix, *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part I, p. 298 (1930).

Wire, as a structural shape, is of very wide application. It would be difficult from a mechanical standpoint to construct a machine for testing wire by repeated axial loading or torsion. The rotating-beam type, however, is particularly adaptable for this purpose. The weight of the specimen, itself, gives added fiber stress which reaches a maximum at the medial transverse plane of the specimen and this is far removed from the ends where contact with the machine gives added and indeterminate local stresses. It is only necessary to make the specimen so long that its own weight adds (at the center of the specimen), stresses which are greater than any possible local clamping stress at the end. The application of this principle is the basis of a long-span machine, described in this paper, for subjecting wire specimens to cycles of determinate stresses. Reduced diameters and polished surfaces of the specimen are avoided. The actual fatigue limit of the wire instead of the endurance limit of the metal can be determined. The effects of various surface conditions, without change in shape, also can be studied.

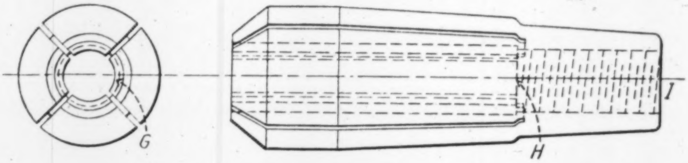

FIG. 2.—Spring Collet for Holding Specimen.

## DESCRIPTION OF THE LONG-SPAN FATIGUE MACHINE

Figure 1 is a diagram of the apparatus which is essentially a rotating-beam machine with a necessarily high deflection of the beam. Bearing boxes, $A$, $B$, of the R. R. Moore type,[1] are attached to both ends of the specimen $C$. The applied load $W$ is divided into two equal parts, which are suspended by flexible cables and helical springs at fixed and equal distances from each bearing box support. One of the bearing boxes, $A$, is supported on ball bearings so that the system will easily adjust itself in a manner analogous to a beam freely supported at both ends. The other bearing box, $B$, is supported on a pivotal axis which is fixed in position. A motor is attached to a base, $E$, which may be pivoted about an axis coincident with the support axis for bearing box *B*. The motor shaft is thus brought into approximate alignment with the bearing box shaft. By means of a tee coupling, *F*, allowance is made for small differences in alignment.

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[1] Report of Research Committee on Fatigue of Metals, Appendix, *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part I, p. 266 (1930).

The specimen is attached at each end to the bearing box shaft or cylinder by means of a spring collet shown in Fig. 2. The end of the specimen is inserted in the collet to the point *H*, and the collet is drawn into the bearing box cylinder with a rod whose threaded end is screwed into the other end of the collet at *I*. As the tapered prongs are drawn into the cylinder, they grip the end of the specimen. The pressure of the prongs on the ends of the specimen necessarily creates zones of local stress concentration, which are markedly increased if the contact plane between the collet and the specimen is irregular. To minimize this effect, the collets are lined with copper bushings, *G*. Copper is ductile enough to permit a distribution of the local stresses over larger areas and so a reduction in their intensities.

A quadrant of light-weight metal strip is attached to each bearing box. The radius is 4 in. and it is fixed so that the axial center coincides with the bearing-box support axis. This maintains a constant lever arm which is independent of the deflection of the specimen. A revolution counter and automatic stop are attached to operate in the usual manner.

## CALCULATION OF STRESS

The maximum fiber stress at the medial transverse plane of the specimen may be computed from the formula:

$$S = \frac{(M_1 + M_2)}{I/c} = \left(\frac{W_1 l}{8} + \frac{W_2 a}{2}\right) \div \frac{\pi}{32} d^3$$

where $S$ = maximum flexural stress, pound per square inch;

$M_1$ = bending moment (in inch-pounds) due to weight of specimen $= \frac{W_1 l}{8}$;

$W_1$ = total weight of specimen in pounds;

$l$ = horizontal distance from face of one grip to face of the other, in inches;

$M_2$ = bending moment (in inch-pounds) due to weights and bearing boxes $= \frac{W_2 a}{2}$;

$W_2$ = force due to weights + effective load due to weight of bearing boxes, in pounds;

$a$ = horizontal distance from bearing blocks to line of pull of weights, in inches;

$I/c$ = section modulus of specimen, for a circular specimen

$= \frac{\pi}{32} d^3$; and

$d$ = diameter of specimen in inches.

## EFFECT OF ROTATIONAL SPEED

It has been repeatedly demonstrated by other investigators that, within limits, the rate at which stress cycles are applied to a metal has no effect on its endurance limit. It is possible, however, that effective stresses may increase if the rotational speed approaches the critical speeds of the system. With the usual types of fatigue testing machines in which a rotating specimen is used, the specimen is short and the total deflections are so small that the fundamental critical speed is far above the usual working speeds. Excessive vibration may be encountered under these conditions when the rotating speed is some sub-multiple of the fundamental critical speed. In order that the weight of the specimen may increase the stress at the center sufficiently, it is necessary to have a long beam and consequently a low critical speed. Obviously, it is desirable to use speeds much above the fundamental critical speed in order to obtain results within a reasonable time.

It is known[1, 2] that the fundamental critical speed of a rotating system is equal to the fundamental vibrational frequency of the system. When the rotating speed is increased slowly, resonant vibrations are set up, which increase in magnitude as the rotating speed approaches the vibrational frequency. When the critical speed is attained, the vibration is limited only by the damping effect of the system. As the rotating speed is increased above the vibrational frequency of the system, resonant vibrations are decreased until the next higher vibrational frequency is approached. It is necessary that the acceleration be small because an appreciable time is required to build up the resonant vibrations. If the rotating speed of a fatigue specimen is within the range of resonant vibrations, indeterminate stresses are set up which invalidate the test results.

Critical speeds higher than the fundamental critical speed may be encountered. As an example, a beam freely supported on both ends vibrates fundamentally with one loop and two nodes, and with higher frequencies two loops and three nodes, three loops and four nodes and so on. Each increase represents a corresponding increase in frequency of vibration. It follows that if the rotating speed of the

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[1] Arthur Morley, "Strength of Materials," Longmans, Green and Co., New York City, 1920.

[2] E. P. Coston, "Strength and Elasticity of Materials, Vol. II," Scott, Greenwood and Son, London, 1925.

beam is increased above the fundamental critical speed, there will be no large forced vibration until the next higher frequency is approached. Above this, there will be a range of no large forced vibration, and so on for each succeeding "over-tone." With increasing stiffness, the higher critical speeds of a beam increase roughly in proportion to the increase of the fundamental critical speed provided the beam is symmetrically loaded and has uniform cross-sectional area. The fundamental vibration frequency of a loaded beam decreases with decreasing stiffness and increasing inertia. Increase in length will decrease the stiffness, and the inertia is influenced by the mass of the vibrating system. It is apparent that small changes in length of the specimen are large factors in critical speed changes, whereas changes in applied load have little or no effect on the critical speed if the loads are not firmly attached to the system.

TABLE I.—DESCRIPTION OF MATERIAL TESTED.

| | CARBON, PER CENT | TENSILE STRENGTH, LB. PER SQ. IN. |
|---|---|---|
| 1. High-carbon steel wire, quenched and tempered, then galvanized by hot-dip process in a commercial plant on production basis.................. | 0.77 | 215 000–230 000 |
| 2. Same wire as No. 1, with zinc coating removed by immersing in HCl (sp. gr. 1.19) to which had been added 0.32 g. $SbCl_3$ per liter HCl[a]............. | 0.77 | 226 000–230 000 |
| 3. Low-carbon steel wire, surface ungalvanized. Previous history unknown........................ | 0.13 | 94 000 |

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[a] See Standard Methods of Determining Weight of Coating on Zinc-Coated Articles (A 90 – 30), 1930 Book of A.S.T.M. Standards, Part I, p. 385.

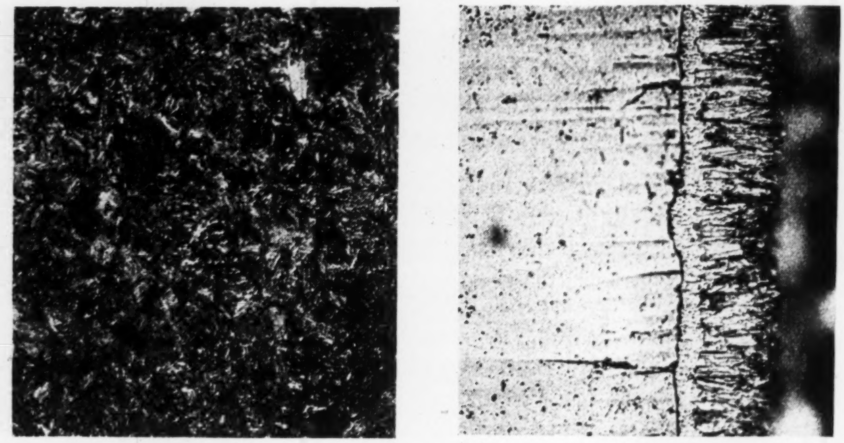
## MATERIALS TESTED

Fatigue limits were determined on the steel wires described in Table I. Microstructures are shown in Fig. 3.

The diameter of the wire specimens ranged between 0.185 and 0.195 in. All specimens were cut 66 in. in length, which insured an added fiber stress due to the weight of the specimen of more than 5000 lb. per sq. in. It was desirable that the added stress be no higher than necessary, because added length decreases the fundamental critical speed and consequently decreases the magnitude of the speed ranges between higher critical speeds. Actually, in the experiments here reported, the added stress at center of specimen ranged from about 5700 to 6200 lb. per sq. in.

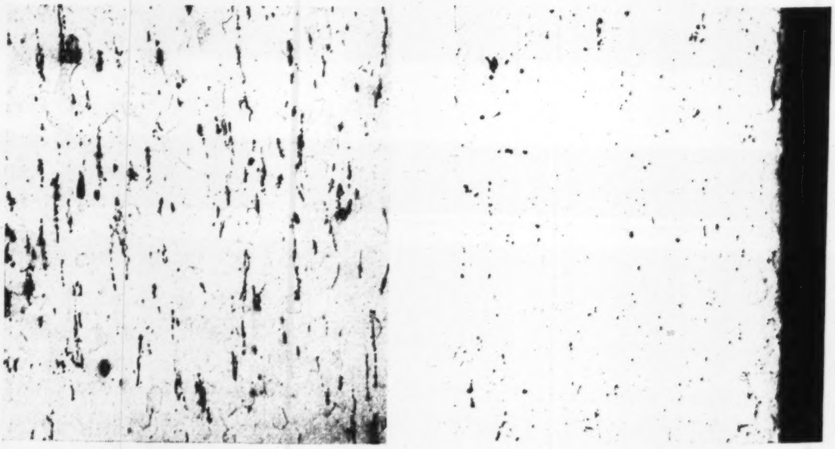
## TEST PROCEDURE

At the start of a series of tests of a material, one of the specimens, which was later discarded, was placed in the machine and loaded at its assumed fatigue limit. The motor was started and quickly attained the speed of 2000 r.p.m. The $\frac{1}{4}$-h-p. motor used had an acceleration great enough to avoid the effects of lower critical speeds. The speed of the motor was slowly increased from 2000 r.p.m. until a critical speed was approached, then decreased until the next lower critical speed was approached. In this way the half way point be-



(*a*) Center of high carbon steel wire (×500). Etched with 2 per cent $HNO_3$ in alcohol.

(*b*) Zinc coating on high carbon steel wire (×500). Etched with chromic acid-sodium sulfate solution.



(*c*) Center of low carbon steel wire (×100). Etched with 2 per cent $HNO_3$ in alcohol.

(*d*) Surface structure of low carbon steel wire (×100). Etched with 2 per cent $HNO_3$ in alcohol.

FIG. 3.—Photomicrographs of Wire Steel.

tween two critical speeds was determined, and used for all tests on the material.

These speeds for the experiments here reported were 2000 r.p.m. for the high-carbon steel wire, heat-treated and galvanized; 1900 r.p.m. for the same wire with zinc coating removed; and 1700 r.p.m. for the low-carbon steel wire.

After failure had occurred, the diameter was measured at a point adjacent to the fracture. Results of the tests were accepted only if the fracture occurred within 12 in. of the center. Over this range the possible decrease in stress from the maximum is less than the possible error due to measurement of the diameter and may be neglected. More than 90 per cent of the test fractures occurred within the 24-in. range, so it was unnecessary to attempt calculations of stress if the point of fracture was near the ends of the specimen.

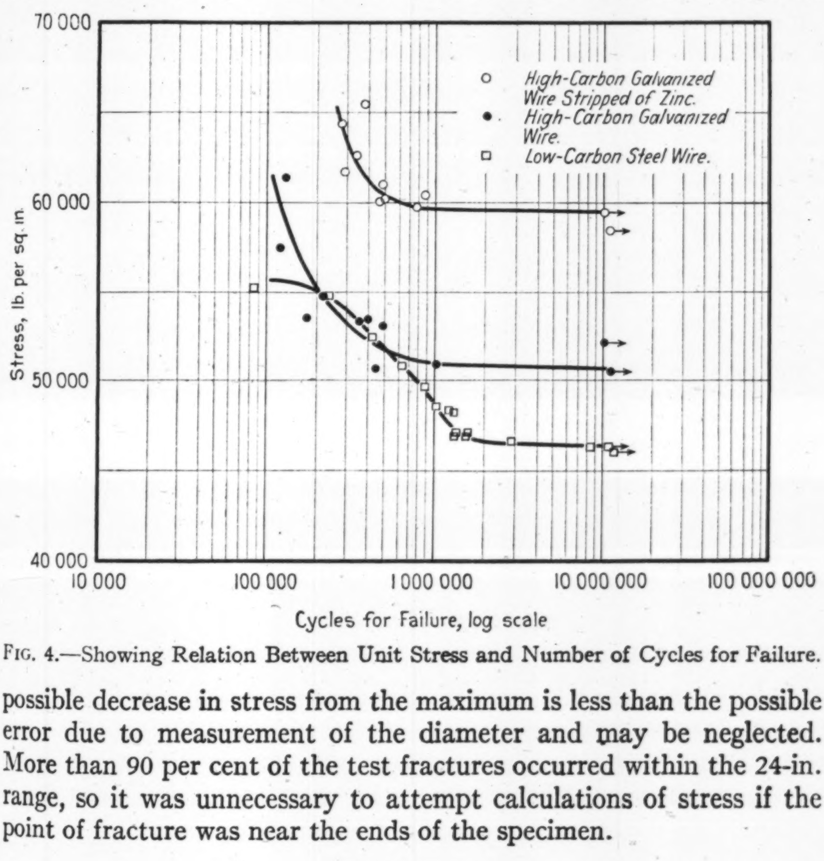


FIG. 4.—Showing Relation Between Unit Stress and Number of Cycles for Failure.

## TEST RESULTS

The results of the fatigue limit determinations are represented as fatigue curves in Fig. 4. The fatigue limit of the galvanized, heat-treated wire was determined to be 50,000 lb. per sq. in. The fatigue limit of the same wire with the zinc coating removed was 60,000 lb. per sq. in. The 0.13-per-cent carbon steel wire showed a fatigue limit of 46,000 lb. per sq. in.

The fatigue limit of the galvanized wire was also determined in the usual R. R. Moore type of rotating-beam machine. Three-inch specimens were threaded at the ends by a swaging process and screwed into threaded adapters. By the cold deformation of the swaging process, it was hoped to secure added strength in the ends, which would more than overcome the loss in strength due to the threads. Only those values obtained on specimens that broke in the unmodified mid-section were accepted. The endurance limit was determined as 50,000 lb. per sq. in. The values obtained with the 66-in. specimens agreed well with this figure.

To show further the difference that might be obtained in some cases between the endurance limit as determined on specimens machined from the wire and that determined on the wire itself, usual fatigue specimens were made from the high-carbon heat-treated galvanized wire and the low-carbon steel wire. Short-length specimens were machined to a shape corresponding to the rotating-beam type of fatigue specimen, with the specimen reduced to a minimum of 0.150 in. at the longitudinal center. The radius of curvature was $9\frac{7}{8}$ in. The surfaces were polished longitudinally on progressively finer and finer emery paper until no scratches were visible to the eye after using 0000 emery paper. Under these conditions, the high-carbon steel showed an endurance limit greater than 100,000 lb. per sq. in. and the low-carbon steel, 56,000 lb. per sq. in.

The endurance limits and fatigue limits were computed on the basis of total diameter of the specimen when tested. In computing the fatigue limit with the galvanized wire, the assumption was made that the coating had no thickness. Obviously, this method of computation fails to give the true maximum fiber stress at the surface of the galvanized wire. Differences in moduli of elasticity for zinc, zinc-iron alloys at the boundary zone and for steel may, and probably do cause large differences in the fiber stress for a given load. But the results are of value in that they show the effect of the zinc coating on the resistance to failure when the wire is subjected to stress ranges resulting from known loads.

The fatigue limit of the stripped heat-treated wire was 20 per cent higher than that determined for the same wire with the zinc coating present. The endurance limit of the specimens of the same wire, which had been machined from an original diameter of 0.195 in. to a minimum reduced diameter of 0.150 in. was approximately 100 per cent higher than that determined for the galvanized wire. Since the structure of the heat-treated steel is more or less homogeneous throughout the section, it is probable that differences in surface condition are responsible for the large differences in fatigue limits.

The endurance limit of the low-carbon steel was 20 per cent higher than the fatigue limit of wire of the same material. This was probably due to the unpolished surface condition of the wire and possibly to surface decarburization in annealing.

The range in diameters of wires which may be tested by the method described is yet to be determined. The minimum possible diameter is dependent on the fixed loads due to the bearing boxes and fixtures as well as friction in bearing box *A*. The torque due to bearing-box friction with the beam loaded was measured under a series of loads. A maximum value of 0.35 lb-in. was determined. A torque of 0.35 lb-in. is equivalent to 280 lb. per sq. in. static shear stress on the outer fibers of a 0.185-in. specimen and therefore may be neglected.[1] Apparently, there is no maximum diameter limit provided the ends of the specimen may be clamped satisfactorily in the bearing boxes.

It is evident that a machine built on the same principles may be used for fatigue testing of metal pipes and tubing. The only modifications necessary are suitable end grips for the specimen and the use of longer specimens in order to obtain added fiber stress at the medial transverse plane. The cost of preparing specimens whether they be of wire, pipe or tubing is negligible.

## SUMMARY

1. The endurance limits determined from test coupons with machined surfaces cut from a structural member cannot be relied upon as the true fatigue limit of the structural member.

2. A method is described for the determination of the fatigue limit of wire in its structural shape and condition. The specimens used are long enough to obtain sufficient increase in fiber stress in the medial transverse plane of the specimen without reducing its cross-sectional area.

3. A galvanized coating has a deleterious effect on the apparent fatigue limit of a high-carbon heat-treated steel wire. The endurance limit of a low-carbon steel is appreciably higher than the fatigue limit of a wire made from the same material.

4. The possibility of using the method for fatigue testing of other structural shapes such as pipes and tubes deserves further investigation.

*Acknowledgment.*—The author wishes to express his appreciation for the cooperation of W. H. Swanger, Metallurgist, Bureau of Standards. His help and advice were indispensable in the preparation of this paper.

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[1] F. C. Lea, "Combined Torsional and Repeated Bending Stresses," *Engineering* (London), August 20, 1926, p. 243.

# DISCUSSION

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MR. R. E. PETERSON[1] (*presented in written form*).—In determining physical properties of materials, the necessity of avoiding stress-concentration has limited the testing engineer to the use of machined specimens. In many cases in practice, unmachined surfaces are subjected to severe stressing, such as, for example, in heavy helical springs in railway service. In such cases machined specimens are not representative.[2] Mr. Shelton has developed a fatigue test of wire which obviates the necessity of machining, and has also shown that specimens machined from the wire are not representative. Some time ago, the Westinghouse Research Laboratories were confronted with a similar problem in comparing the fatigue strengths of centerless ground stock and hot-rolled stock for use in helical springs. The stock provided was 1 in. in diameter throughout. The requirements were that the surface be undisturbed and that a critical section be formed with no stress concentration due to gripping. A cantilever fatigue specimen was made as shown in the accompanying Fig. 1, using a special round-nose reamer to produce the bore contour. The tests are not completed as yet, but it is of interest to know that failure occurs in the desired manner, that is, at an unmachined surface without appreciable stress concentration.

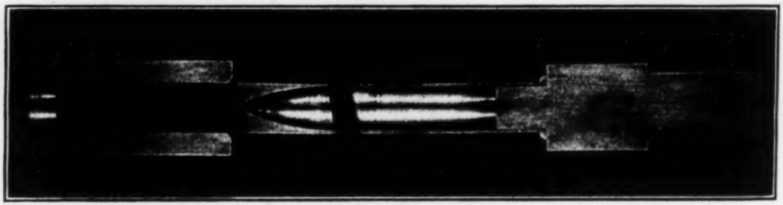

FIG. 1.—Cantilever Fatigue Specimen.

The statement is made in the paper that with the usual types of fatigue testing machines in which a rotating specimen is used, the specimen is short and the total deflections are so small that the fundamental critical speed is far above the usual working speeds. It is true that the natural frequency of the specimen itself is far above the operating frequency; but in this case one must consider the natural frequency of the system as a whole. Analytically, the system may be considered as having one degree of freedom, the elasticity being represented in the specimen, and the mass by the suspended weight. The frequency is:[1]

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[1] Manager, Mechanics Division, Research Laboratories, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[2] Discussion by G. M. Eaton of paper by M. F. Sayre, *Transactions*, Am. Soc. Mechanical Engrs., APM - 51 - 24, p. 304 (1929).

$$f = \frac{1}{2\pi}\sqrt{k/M}$$

where $f$ = frequency in cycles per second, $k$ = spring scale in pounds per foot and $M$ = mass (lb. sec.$^2$ per ft.). The natural frequency is seen to vary inversely as the square root of the suspended weight. Computations show that the natural frequency of the system is not, in general, far above the operating frequency, but on the contrary is often below the operating frequency. In shutting down a cantilever machine, unless the weights are drawn up with a nut, severe vibrations occur as the machine coasts down through the critical speed. The frequencies computed by the above formula may also be readily checked with a vibrograph.

It is also stated in the paper that excessive vibration may be encountered when the rotating speed is some sub-multiple of the fundamental critical speed. This is contrary to theoretical and experimental observations.[2,3] Resonant peaks are not obtained when operating below the critical speed unless the cross-section is non-symmetrical.

Ideas which are essentially new are few and far between in the field of testing of materials. The author is to be congratulated for developing a new and ingenious test method.

MR. H. F. MOORE.[4]—For the ingenuity of this method of fatigue testing there can only be commendation. It has been shown that the long specimen used can be made to run smoothly.

The ingenuity of the method makes me raise a question. Would it be possible to secure the slight additional pull at the center necessary to make the higher stress in the middle in the grips by *magnetic* methods, say by means of a magnet resting on a weighing scale to measure pull?

In connection with the question of vibration of specimens, one phenomenon has been quite evident in fatigue tests at the University of Illinois. The surprising result for nearly all materials so far tested, with the exception of cast iron, is that a slight vibration synchronism with the speed of rotation produces slight effect upon the endurance limit. Specimens which accidentally began to vibrate after the test had started frequently ran out along the normal *S-N* curve. Such a vibration in tune with the rotation of the machine does not change the *range* of stress to which the specimen is subjected but does change the maximum and minimum values, and a slight change of the range of partially reversed stress for a specimen does not make very much change in the endurance limit.

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[1] S. Timoshenko, "Vibration Problems in Engineering," p. 1, D. Van Nostrand Co., New York City, 1928.

[2] Frith and Buckingham, "Vibration in Engineering," p. 36.

[3] S. Timoshenko, "Vibration Problems in Engineering, p. 29, D. Van Nostrand Co., New York City, 1928.

[4] Professor of Engineering Materials, University of Illinois, Urbana, Ill.

I should like to call attention to two rather striking results of these tests. I refer to what is usually called the endurance ratios of the specimens, the ratio of the fatigue limit to the tensile strength. For the galvanized wire and for the wire with galvanized coating removed, these ratios are very *low*, about 26 per cent with the galvanizing removed, and 22 per cent for the galvanized wire. I should expect wire of that tensile strength to have an endurance ratio in the vicinity of 40 per cent.

MR. A. V. DE FOREST.[1]—I also wish to add a word of praise for this extremely ingenious method of testing wire in fatigue. That has been a difficult thing to do at best, and this very unexpected solution seems almost too good to be true.

One of the elements in this test, I think, ought to be stressed. The fact is that in this test there is a very much longer length of material under uniform load than in any fatigue test of the normal type with which I am familiar. If one is dealing with the material of a specimen, a test of the fatigue properties of a particular finished product are not important, but in the case of wire it is not the fatigue properties of the material but the fatigue properties of the surface that are important, and the chances of getting a damaging notch in a length of 4 ft. are very much better than in a length of 0.4 in., so that this test is equivalent, with a 4-ft. gage length, to a very great many tests conducted on the usual scale.

This consideration brings up the point that we know very little about the fatigue properties of materials in the state in which they are used, because in the normal fatigue machine, in order to get results that will hang together, the surface is taken off and a nice polish is put on. That is satisfactory for the fatigue investigations of metals, but it means nothing as to the fatigue investigation of parts, which are affected by the mill conditions of surface which obtain on the finished article. This test attacks both ends of the problem, a requisite which is becoming better appreciated every day.

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[1] Consulting Engineer, American Chain Co., Inc., Bridgeport, Conn.

MR. R. R. MOORE[1] (*presented in written form*).—Mr. Shelton has very ingeniously applied the rotating-beam principle to the fatigue testing of small wire which is a very valuable contribution to fatigue testing.

The results of his tests are extremely interesting. From the test results given in Table I of the paper the writer has computed the endurance ratio, that is, the ratio endurance limit to tensile strength, for the high-carbon galvanized wire and for the same wire after the galvanizing had been removed, and finds that these values are about 0.23 and 0.26, respectively. Tests result quoted on page 211 of the paper for specimens which have been reduced in the center by removing the outer surface give an endurance ratio of 0.45 for the same high-carbon wire. The extremely low results obtained on the galvanized wire and after removing the coating, are less than has ever been reported on any steel.

Mr. Shelton has drawn the conclusion that the extremely low endurance limits are due to the galvanizing but he does not specify what particular phase of the galvanizing process is responsible. In view of the lack of information as to the heat treatment used, and the absence of careful microscopic examination at higher magnifications, the writer does not believe that this conclusion is justified. It is suggested that the removal of the outer layers of the wire, in preparing the machined specimens may have eliminated material that was adversely affected during cold drawing and heat treating operations and thus resulted in a much higher endurance limit on the prepared specimens.

It should be noted that no endurance curve is given to show the actual tests on reduced sections; the results are merely quoted in the text. As these test results are rather abnormal, it is highly important that these endurance curves be included in the paper, and it is suggested that Mr. Shelton include this information in his closure to this discussion.

The matter of heat treating high-strength steel wire is extremely important in a number of industries, particularly bridge wire, and the manufacture of valve springs for automotive and aircraft engines. At the present time there is considerable discussion as to which is the best type of wire for automobile and aircraft valve springs, that is, whether it should be heat-treated or a hard-drawn wire of the music wire or piano wire type. The results given by Mr. Shelton have an important bearing on that particular phase of the subject. I am somewhat doubtful as to whether the galvanizing alone is responsible for the damage, in view of the fact that the specimens are high-carbon steel, quenched and drawn.

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[1] Chief Metallurgist, Wright Aeronautical Corp., Paterson, N. J.

MR. T. S. FULLER.[1]—I have been particularly interested in the paper by Mr. Shelton because it corroborates results reported by our laboratory some two or three years ago on galvanized fatigue specimens. A very considerable lowering of fatigue strength due to galvanizing was found, and was attributed to the presence of the brittle iron-zinc layer, which as the test specimen is deflected produces cracks with accompanying stress concentration effects which cause the apparent lowering of the strength. I think there is no question but that we must expect a considerable lowering in endurance limit of galvanized specimens.

MR. J. B. KOMMERS.[2]—I was impressed by the ingenuity shown in the design of the machine described by Mr. Shelton. One of the beautiful features of the scheme is the very simple specimen that can be used and the inexpensiveness of that specimen. The fact that specimens may have to be tested in the condition in which they are fabricated is brought out in the paper itself, and was also brought out by some experiments in England on leaf springs, in which the completely fabricated spring showed endurance limits only 25 to 40 per cent of the material from which the springs were manufactured. These results were investigated and it was found that the heat treatment of the leaves seemed to decarburize the surface, and that the surface condition was responsible for the lowering of the endurance limit of the completely fabricated spring.

MR. J. R. FREEMAN, JR.[3]—I should like to ask whether Mr. Shelton has tried out swaged threaded-end wire specimens in the Haigh machine. In the earlier days of the tests of wire on the rotating-beam machines we first tried to use a specimen of the same type as they have succeeded in doing without any reduced test section. In order to hold the specimen in the machine we used swaged threaded ends. The cold working from the swaging operation apparently increased the strength of the wire sufficiently so that the break did not occur at the threads. I think the method Mr. Shelton developed is superior to the swaged thread method but it would not be applicable to the Haigh machines, whereas the swaged threaded-end specimens might work out in the Haigh machines.

MR. W. H. SWANGER.[4]—We have obtained fairly good results on the R. R. Moore machines with 3-in. lengths of wire specimens that had threads swaged on to each end for a length of about one inch. The greater proportion of breaks occurred in the middle inch of the specimens. However, we have not been able, so far, to hold such specimens in the Haigh machines, with the stresses ranging from compression to tension. Possibly they might hold if the Haigh machines were operated with a stress range in tension only.

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[1] Metallurgist, Research Laboratory, General Electric Co., Schenectady, N. Y.

[2] Professor of Mechanics, Engineering College, University of Wisconsin, Madison, Wis.

[3] Technical Department, American Brass Co., Waterbury, Conn.

[4] Metallurgist, U. S. Bureau of Standards, Washington, D. C.

Mr. Shelton obtained the same value for endurance limit of the steels he has so far tested by using 3-in. lengths of threaded-end (threaded by swaging) specimens as by using 66-in. lengths on his long-span machine. He has not been able to run 66-in. lengths of cold-drawn bridge wire on his long-span machine. This wire has a permanent set which cannot be removed without deformation drastically exceeding the yield point. After such deformation, the properties of the wire are altered and the endurance limit of the wire as drawn, or as actually put into use, would not be obtained.

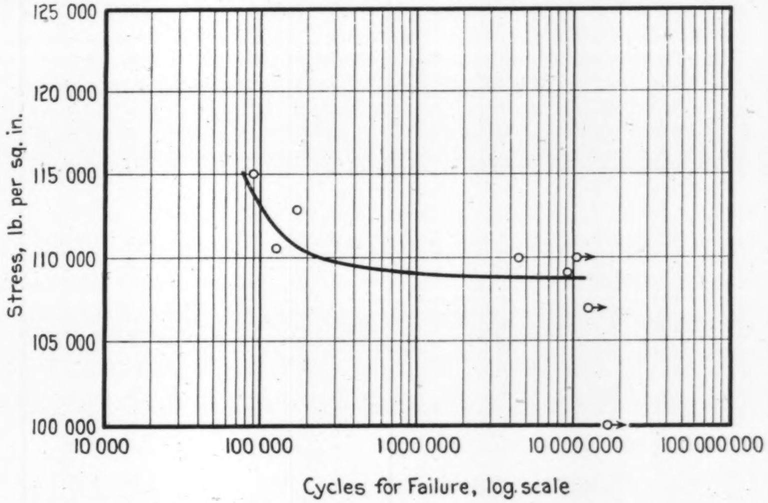

FIG. 2.—*S-N* Diagram for 3-in. Specimens of High-Carbon Steel Wire, Reduced and Polished.

MR. S. M. SHELTON[1] (*author's closure by letter*).—At Mr. R. R. Moore's suggestion, the *S-N* diagrams showing the results obtained with 3-in. length specimens machined from the low-carbon steel wire and from the high-carbon steel wire, for which the endurance limits on the unreduced section, 66-in. lengths, are given in Fig. 4, are given in the accompanying Figs. 2 and 3.

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[1] Junior Metallurgist, U. S. Bureau of Standards, Washington, D. C.

The specimens were reduced in the mid-section from a nominal diameter of 0.2 in. to a minimum diameter of 0.150 in., using a radius of curvature in the reduced section of 9⅞ in. The machined surfaces of the specimens were polished longitudinally with progressively finer and finer emery papers until no scratches were visible to the eye after using 0000 emery paper.

Figure 2 shows the results obtained with the high-carbon steel wire specimens, and Fig. 3 shows those obtained with the low-carbon steel specimens. The values for the endurance limits of these two materials reported in the paper had been taken from these curves.

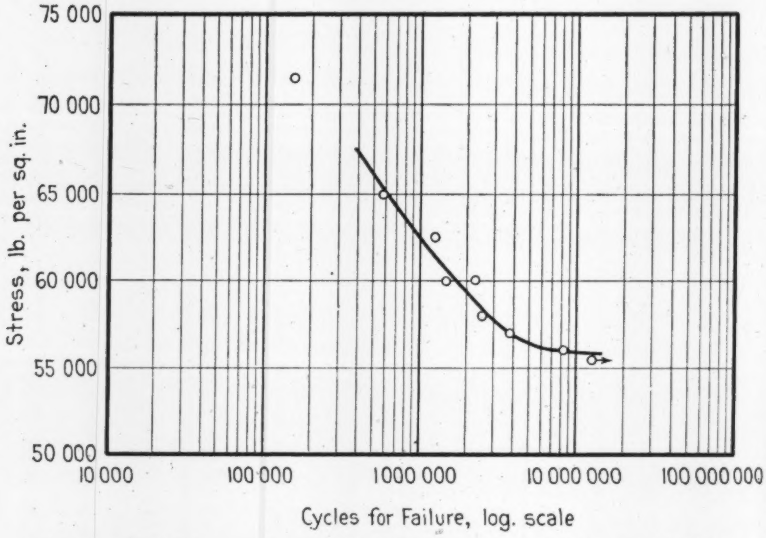

FIG. 3.—*S-N* Diagram for 3-in. Specimens of Low-Carbon Steel Wire, Reduced and Polished.

This paper is primarily a description of a newly developed method of test, and consequently a well-rounded study of the fatigue properties of various types of wire was not included. However, studies of the effect of the galvanizing process as well as other treatments on the endurance properties of wire are being continued at the Bureau of Standards and the results of these investigations will be reported in the near future.

# A SEVEN-TON 50-CYCLE FATIGUE TESTING MACHINE

By B. P. Haigh[1] and T. S. Robertson[1]

## Synopsis

A new model of the Haigh fatigue-testing machine, redesigned in the light of several years' experience is described. The new model has been designed particularly for tests of an engineering nature, on scale models of parts of machines and structures, that is, riveted or welded joints and screwed connections, but is serviceable also for testing specimens of standard shape. It is capable of 22,000-lb. maximum load with ranges up to 14,000 lb. with a frequency of 3000 cycles per minute. The loads are applied as direct pull and push, and simple devices insure that no bending or torsion is present during the test.

## Fundamental Principles of Action

In 1911, in the discussion that followed a paper before the Institution of Mechanical Engineers in London, three different types of electromagnetic fatigue testing machines were described in public for the first time. The first small Haigh machine, constructed in 1910 in the University of Glasgow, was followed in 1913 by a larger model built in the Royal Naval College, Greenwich; and a later design of the same size, giving a range of load of 3000 lb. with a frequency of 2000 cycles per minute was described in 1921 in *The Engineer* (London). A large number of machines of this well-known design manufactured by Bruntons, of Musselburgh, Scotland, are installed in laboratories throughout the world; and the general use of the machine by different authorities has contributed much of value to current knowledge of fatigue of metals.

The work undertaken in this standard machine has been chiefly of a metallurgical rather than an engineering character. The range of loads is intended and is suitable for testing specimens $\frac{1}{4}$ in. in diameter in strong steels, or larger specimens of weaker metals, but is hardly sufficient for testing complex models of rivetted or welded joints or other fastenings used in structures and machines. The object of the present paper is to describe a larger model that opens up a wider field of investigation. The new model works with ranges

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[1] Royal Naval College, Greenwich, London, England.

of loads up to 14,000 lb., giving crest loads up to 20,000 lb. with a frequency of 3000 cycles per minute, corresponding to over four million cycles per working day.

Examples of this larger model, manufactured by Bruntons and first exhibited in operation in 1929 at the Exhibition at Newcastle, England, are installed in several laboratories in England. Figure 1 shows the new machine in the Engineering Laboratory of the Royal Naval College, Greenwich. The machine stands 8 ft. in height and rests on a floor without any special foundation. The control and measuring instruments are mounted on a panel to the right, and the necessary two-phase current is supplied from a special motor-generator on the left.

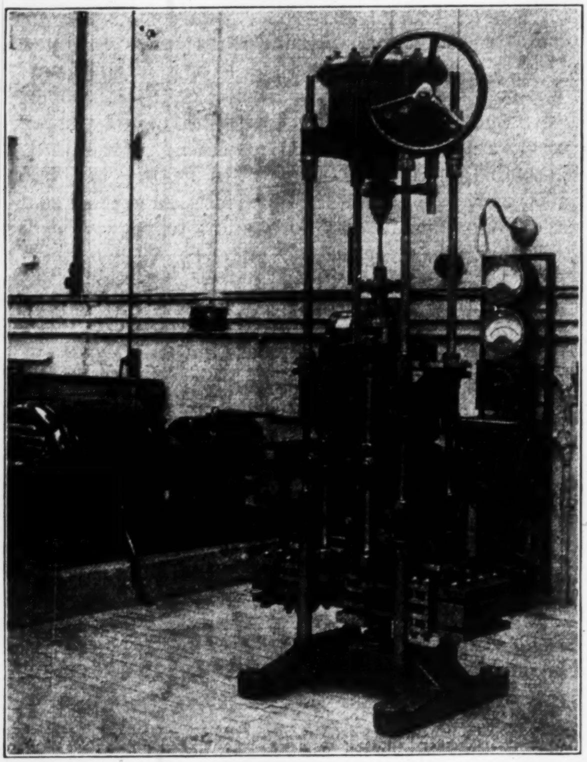

FIG. 1.—Showing a Machine Installed in the Royal Naval College, Greenwich.

The outstanding characteristic of the Haigh machine is the use of a two-phase alternating-current magnet in combination with a

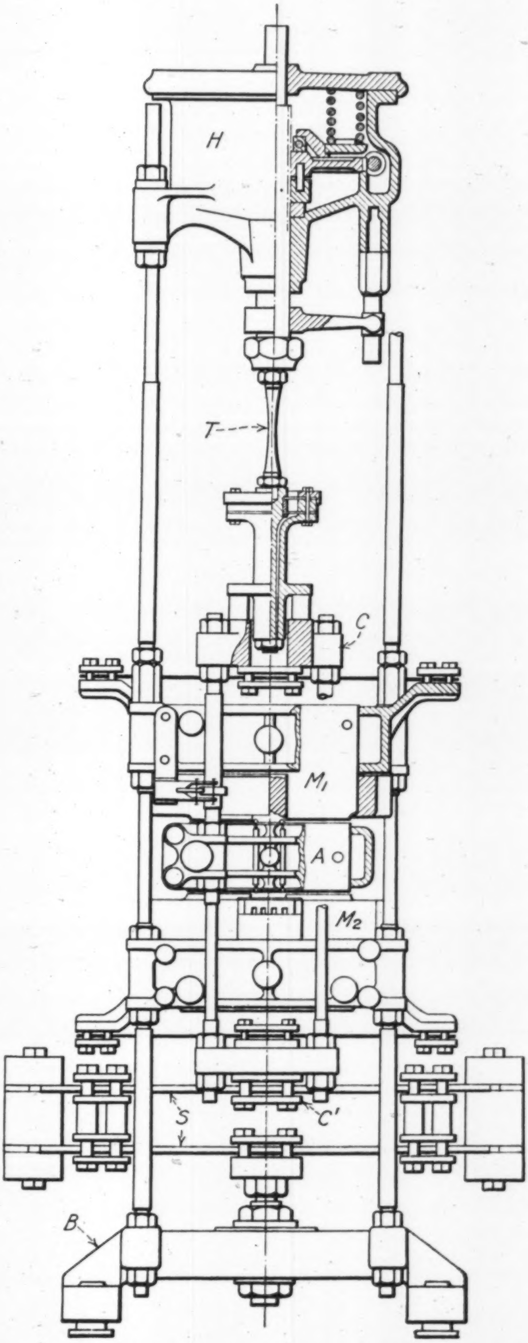

FIG. 2.—Sectional View Showing Test Specimen in Position with the Weighing Bar.

spring that fulfills two distinct functions. The general arrangement of the machine is shown diagrammatically in Fig. 2.

The upper end of the test specimen *T* is gripped in a holder that forms part of an adjustable head *H* supported on columns that rise from the base *B* and carry two U-shaped laminated magnets $M_1$ and $M_2$ pulling on opposite faces of a single laminated armature *A*. The lower end of the test specimen is connected to the armature by stout bars from a cross-head *C*, and these same bars are extended downwards to the spring *S* which is connected also to the base *B*.

The machine may be regarded as comprising two distinct groups of parts constituting two members that are coupled elastically and are capable of relative motion in the vertical direction. The first group, comprising the armature, the two cross-heads *C* and *C′* and the connecting bars, may be termed the vibrating element; while the second group, comprising the head *H*, the two magnets and the base *B*, may be termed the stator, although actually being mounted on springs under the feet of the base, it vibrates reciprocally against the first group. The two rigid groups are coupled elastically by the test specimen *T* and also by the spring *S*.

The primary function of the spring *S* is to support the weight of the armature and other parts of the vibrator so that the test specimen is free from any steady load, or to apply any specified steady load acting as a pull or push in the test specimen in addition to the alternating load applied by the magnets. The spring is connected to the base *B* by means of a spring fitted with micrometer adjustment; and by adjusting this screw upwards or downwards before the test is started, any desired steady load may be applied at will.

The secondary function of the spring *S* is to compensate the force absorbed in accelerating the massive parts of the vibrating system in their small amplitude of rapid motion. When the stiffness of the spring is correctly adjusted, a very simple operation in practice, the force of the magnets is wholly transferred to the test specimen, and the range of load can then be measured electrically in a convenient and effective manner.

## MECHANICAL DESIGN

Although the large machine was designed in the light of several years experience of the working of the smaller model, certain interesting difficulties presented themselves in obtaining the greater range of loads in combination with the higher frequency that was regarded as desirable for expediting tests. According to a simple application of the well-known and exceedingly serviceable principles of mechanical similarity, first enunciated in 1875 by Professor James Thomson of Glasgow, a geometrically similar machine giving a fourfold range of load with the same factors of safety would weigh eight times as much and should operate with equal success at half the frequency of its smaller prototype. The major problem in design was, therefore, to increase the frequency of operation from 2000 to 3000 cycles per minute without sacrifice of accuracy, instead of accepting the reduction from 2000 to 1000 cycles per minute.

The first step was to make the operation of such a large machine tolerable in a laboratory in which other experimental work was in progress. Instead of mounting the stator base on a large mass of concrete, as had been the practice for the smaller model, the base was supported on four springs arranged within the hollow feet of the casting. This change has proved thoroughly acceptable. The machine communicates no vibration to the floor, and conversely, is unaffected by any vibration that may occur in the floor. The stiffness of the springs and the spread of the base are proportioned to render the machine stable on its elastic support. Fixing screws are provided for locking the machine when adjustments have to be carried out, but these locking screws are always loosened before the machine is operated and the machine is then readily rocked by hand on its spring supports.

Little difficulty was experienced in making the machine strong enough to stand the specified 16,000 lb. range of loads with a 22,000 lb. crest load, although the scantlings required to stand the severe conditions are comparable with those of an ordinary tension testing machine of at least double the nominal capacity. It may be of interest to note that mild steel was used in many essential parts, such as the main columns, in preference to alloy steels because, although any saving in weight would have been advantageous, rigidity was held to be even more important.

Figure 2 illustrates the arrangement and certain important details of the upper part of the machine. The magnets and armature are of "Stalloy" stampings located by fitted bolts arranged to transfer the load as directly as possible to the vertical columns of the vibrator and stator. Additional insulated bolts hold the stampings under pressure; and, in order to insure accuracy of alignment, much of the machining of the castings is performed after the stampings have been firmly clamped in their final positions. The cross-head castings are of steel.

The design of the adjustable head of the machine is governed by a number of requirements that fall into two groups: first, the upper end of the test specimen must be gripped firmly without backlash in correct alignment with the rest of the machine; and second, the mechanism must be arranged so that the test specimen can be raised or lowered slightly during the continuance of the test to keep the armature mid-way between the pole faces of the magnets. During the first hour of a test, this adjustment is used occasionally to meet expansions due to changes of temperature; and subsequently, it may be required if the test specimen yields at all plastically. It is used also before and after each test when the specimen is being inserted or removed; but the method of gripping is such that only slight adjustment is required.

The test specimen is screwed to an adaptor forming part of a central member guided by the top and bottom castings of the head of the machine. This central member is supported by a nut that rests on a bronze ring fitted into the lower casting, and is pressed down on this support by a spring load that exceeds the greatest crest load of the machine. The spring load, acting constantly at all times, is applied to a plate resting on a ball thrust bearing on the top of a second nut meshing with the square thread of the central member. The resultant load on the bronze ring is the sum of the spring load and the force in the test specimen, and consequently varies during the cycles of stress.

The two nuts on the central member are coupled together by pins, so that they rotate together when the central member has to be raised or lowered; and the pins are located so that a certain clearance remains open between the two nuts. The rotation is effected by a hand wheel and worm gear of high gear ratio; and the friction introduced by the bronze ring below the lower nut suffices to insure that the hand wheel does not slack back during the continuance of a test.

The strong spring used for loading the test specimen and for compensating the inertia of the moving parts comprises six ground strips arranged in two groups of three. The springs are loaded as beams, clamps being arranged so that the span can be varied to suit the frequency of operation. The several clamps bear on the springs through small cylinders of silver steel. As shown in Fig. 1, the extremities of the springs carry large masses of iron, the function of which is to modify the frequency of local natural vibration so that no resonance occurs under working conditions.

The routine of test employed for adjusting the span of the spring to suit the frequency of operation is exceedingly simple. The machine is run with a very low voltage supply, only 3 or 4 volts instead of

400 as would be required for full load, and the frequency of the supply current is carried until resonance occurs when no test specimen is in the machine. The span of the springs is then adjusted until this resonating frequency coincides with the desired frequency of operation. It is important to note that, although resonance occurs in the manner described when no test specimen is fitted in the machine, the conditions are completely altered when the stiff test specimen is inserted. Under normal conditions of operation, therefore, the working frequency is far below the value at which resonance might become evident.

## THE STRESSES IN THE TEST SPECIMEN

In order to test metal under definite and ascertainable ranges of stress, in a scientific manner, the machine has been designed to embody three important characteristics as follows:

1. The method of gripping is such that the test specimen is free from any initial strain of bending or torsion.
2. The relative motion between the two groups of rigid parts is such that the applied loads strain the test specimen longitudinally without bending or torsion.
3. A sensitive weighing bar is arranged in series with the test specimen so that the maximum and minimum loads applied during the cycle can be observed directly by a delicate strainometer using the deflection of a beam of light.

In Fig. 2, the lower end of the test specimen is fastened to the upper end of a weighing bar fitted in a case that is formed with a broad flange; and this flange is bolted to the upper cross-head of the machine on three distance pieces. When a test specimen has to be removed from the machine, these bolts and distance pieces are the first items to be slackened off; and when a new test specimen is being inserted, the same bolts and distance pieces are the last to be tightened up. The distance pieces serve not merely to give space for the removal of the test specimen and measuring-bar case, but also as sensitive gages. When the machine is adjusted so that the gages slip into place nicely, the operator knows that the test specimen is free from any initial strain in bending. The bolt holes in the flange are clearing holes to insure that there is no initial torsion.

Figure 2 illustrates the two sets of guide springs that are used to insure that the relative motion between the two groups of vibrating parts is exactly in the direction of the axis of the test specimen, insuring that the applied load produces no bending moment in the test specimen. The upper set of springs is shown in part in Fig. 2;

and the general arrangement of the guide springs is shown in Fig. 1. The guide springs insure that the relative motion is purely translational, without any angular motion. No disadvantage is incurred in making them exceedingly stiff for this purpose, because any stiffness added in the guides saves an equal amount of stiffness in the springs *S* provided for compensating the inertia of the armature.

No lubricated slides are used for guiding the relative motion. The conditions are such that lubricated slides, no matter how carefully they might be fitted, would soon suffer abrasion, resulting in backlash, noise and irregular loading. The action of the guide springs has been found exceedingly effective. The operation of the machine is almost perfectly silent even on full load, only the faintest hum arising from the tightly-packed cores of the magnets.

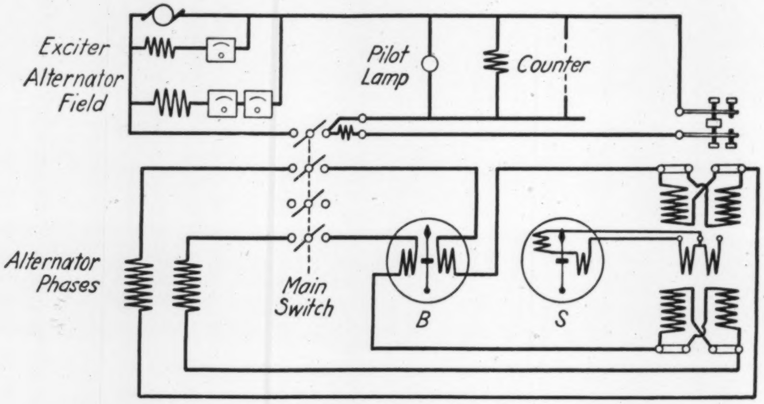

FIG. 3.—Diagram of Electrical Connections.

To insure that the test specimen remains free from torsion, in spite of occasional adjustments to keep the air gaps equal, the central member is provided with means to prevent its turning even slightly. In the smaller models of earlier design, a simple feather was used to prevent rotation of the central member of the upper head; but in the larger model, rotation is prevented by means of a lever that carries bronze blocks meshing with a vertical guide.

The weighing bar used in series with the test specimen provides a means of ascertaining at any time the actual values of the maximum and minimum loads supplied to the test specimen. It is used in the first place for standardizing the stressmeter that indicates the range of stress electrically, and for standardizing the micrometer scale of the loading springs; but it can be used at any time as a direct check or for other purposes, for example to reveal the precise harmonic form of the wave of stress. The weighing bar is of "Delta" metal of high elastic strength and is fitted in a case that carries also a couple of flat plates that act as elastic guides to prevent lateral displacement. The optical system is arranged on the principle of an autocollimator; that is, light from the horizontal filament of a low-voltage lamp passes twice through the same lens and is focussed on a clear glass scale that is examined through a fixed-focus magnifying glass.

## ELECTROMAGNETIC DESIGN

Although the electrical connections of the machine are simple, the theory of operation comprises several points of interest to electrical engineers. The machine may be described as a two-phase choking coil with variable air gap and is notable for the very small power consumed in operation.

The complete diagram of connections is shown in Fig. 3. A small alternator supplies two-phase current to the primary windings of the upper and lower magnets $M_1$ and $M_2$ of the testing machine. Each magnet has two coils that are connected in parallel in normal service; and on full load, the machine takes 400 volts per phase at 25 cycles, giving 50 cycles per second mechanically. The full-load current is 22 amperes per phase with a very low power factor. The range of load is regulated by adjusting the voltage of supply. Ranges less than quarter-load are obtained by connecting the magnet coils in series instead of parallel.

In view of the increase of size and frequency of operation specified, the "apparent," nearly wattless, input to the testing machine deserves consideration. The volt-amperes input is directly proportional to the product of three factors, namely, the range of load, the width of air gap of the magnets and the frequency of operation. It follows that the increase of working frequency from 1000 to 3000 cycles per minute has the effect of trebling the nominal rating of the alternator required to supply current to the machine. The rating of a given alternator frame and winding, however, also increases in direct proportion to the speed of rotation—which determines the frequency—or even somewhat faster. As a consequence it follows that the increased output of the testing machine, measured by the number of test specimens that can be broken in a given time, has been trebled without any increase in the size or cost of the alternator or testing machine.

The balancing ammeter $B$ serves as an exceedingly sensitive extensometer, revealing any slight change in length in the test specimen and indicating to the operator of the machine whether the gearing of the upper head should be adjusted to raise or lower the armature to its normal working position mid-way between the pole faces of the upper and lower magnets.

This balancing ammeter operates on a principle which, although probably novel when it was introduced in 1913, has been employed successfully in a wide variety of other applications—for example, in measuring the strains in bridges and aircraft, and the deflections of vibrating turbine disks and automobiles. The two phases of the main circuit pass through two separate coils that pull in opposite directions on two pieces of soft iron mounted on a single axle in the meter. When the air gaps above and below the armature are equal, the testing machine takes equal currents in its two phases, and the needle of the balancing ammeter points to its central zero no matter what the value of the currents may be. But if the test specimen elongates at all, reducing the lower gap and opening the upper gap, the currents in the two phases change correspondingly, and the needle of the instrument is deflected to the side marked "raise" or "lower." The scale of the meter is graduated in amperes so that the current in either phase can be read if the other phase be disconnected.

In operating the machine, very little attention is required to keep the armature in its normal position. From time to time during the first hour, the hand wheel may be adjusted slightly to compensate for the warming of the machine and test specimen; and the needle will thereafter remain central unless the test specimen yields plastically.

The diagram of connections shows also the electrical stressmeter that is used for indicating the range of load in the test specimen. This instrument also is of the soft-iron type, and is akin to a voltmeter in its construction. It is supplied—through a choking coil instead of a series resistance—from a "secondary" coil wound around the armature of the testing machine.

The action of the stressmeter circuit is such that the reading of the instrument is a function of the flux in the testing machine and independent of the frequency of operation. The instrument gives full-scale reading with 25 volts at 25 cycles, or with 20 volts at 20 cycles, and so on for other combinations corresponding to the same maximum flux. The armature of the testing machine is wound with two coils connected in series. Only one of these is used in normal operation although the two together, giving a total of 14 turns in series, are used for ranges below quarter-load in order to obtain larger deflections on the meter. The scale of the meter is uniformly divided, and is calibrated by reference to the readings of the weighing bar described in the preceding section.

When the test specimen breaks on the conclusion of a test, an automatic tripping gear opens the main switch between the alternator and testing machine; and this same switch simultaneously opens the circuit that supplies direct current from the exciter to hold on the counter and feed the pilot lamp and any other circuits that may be attached. The main switch contains also a fourth set of contacts that can be applied, if so desired, to stop any motor that may be employed to drive the alternator.

The tripping gear is of a simple and effective type and readily adjusted so that the machine stops before the crack completely severs the test specimen. This is desirable as it preserves the crack in a condition that makes visual and microscopical examination of the broken faces more fruitful as a means of ascertaining the origin of the fatigue failure.

## INFLUENCE OF HIGHER HARMONICS IN THE LOAD CYCLE

Although the results of fatigue tests in general appear to indicate that the phenomena of fatigue depend directly on the extreme values of the stresses imposed during the cycles of loading and are little if at all affected by the way in which the stresses change between these extremes, it is proposed to consider the circumstances and degree in which higher harmonics may occur in combination with the fundamental harmonic action in the electromagnetic machine. It will be shown in particular that the attainment of a simple harmonic cycle simplifies the use of the stressmeter already described. When higher harmonics occur in the cycle of loading, the calibration of the stressmeter changes with the elastic stiffness of the test specimen.

The higher harmonics in question arise primarily in the wave of e.m.f. of the two-phase supply current, and are reproduced with changes in the waves of flux and pull of the magnets. The results of a simple mathematical analysis show that odd harmonics occuring in the e.m.f. wave produce even as well as odd harmonics in the stress wave; and, also, that the harmonics in the stress wave are relatively smaller than those in the e.m.f. wave.

Although the alternator used for supplying two-phase current to the testing machine has been specially designed for the purpose, and although the windings of the armature and field circuits were chosen to give the best possible waves with lagging currents, traces of higher harmonics are perceptible in the e.m.f. wave delivered to the machine. Under normal operating conditions, the wave of loading is an almost perfect harmonic curve free from higher harmonics.

Under abnormal operating conditions, however, even traces of higher harmonics present in the cycle of loading can be magnified by actions of resonance; and as such resonant actions interfere with the calibration of the stressmeter, it appeared desirable to design the machine so that they could not resonably occur in practice. This was attained by adjusting the masses and stiffnesses of the parts of the several groups of parts; for example, two large masses of rolled steel are bolted to the ends of the loading springs.

In testing specimens or ordinary dimensions, the weighing-bar unit is connected in series with the test specimen as shown in Fig. 2; but when long springy specimens or models are being tested, the weighing bar is commonly omitted, not only in order to obtain more working space but also to avoid undue magnification of traces of higher harmonics in the cycle.

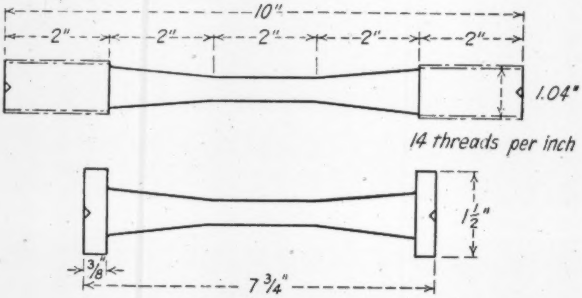

FIG. 4.—Standard Test Specimens.

The spring $S$ is adjusted in stiffness so that when no test specimen is connected between the vibrating parts the frequency of natural vibration of the system is equal to the intended frequency of operation in subsequent tests. When the test specimen and weighing bar are connected, the frequency of natural vibration is raised in the ratio of the square root of $\frac{E_s}{E_w - E_T}$ where the symbol $E$ is used to denote extensibility. The essential condition for avoiding resonant magnification of higher harmonics is that the modified value of the natural frequency shall not be an integral multiple of the operating frequency. When short stiff test specimens are used with the weighing bar in series, the value of the quotient $\frac{E_s}{E_w - E_T}$ is somewhat less than 9. Longer test specimens are tested with the weighing bar in series so long as the quotient value lies well above 4. In testing still longer specimens, or models that deflect more readily, the weighing-bar unit can be either omitted to keep the quotient value greater than 4, or retained to bring the value below 4. When the alternator is driven by a motor with speed regulation, the position of the resonating speed can be ascertained by observation and be readily avoided by working at a modified operating speed.

## STANDARD TEST SPECIMENS

Although the machine is capable of testing specimens of any reasonable form, certain standard sizes of test specimens have been adopted for convenience. Figure 4 shows two standard forms with screwed and cheese-headed ends respectively. The mid-part of the test specimen, 2 in. in length, is cylindrical with gradual transition curves; and the ratio of reduction is sufficient to insure that fracture occurs in the mid-part, even in metals that are especially liable to the influence of stress-raisers. The specimens are turned on centers, great care being exercised to insure that the ends are true with the centers, and the surfaces of the mid parts and transition curves are finished with care to insure that the results obtained are optimum values.

In order to compare different metals in their resistance to stress raisers, a small hole $\frac{1}{32}$ in. in diameter is drilled diametrically through the mid-part of the standard test specimen described.

## SPECIAL FITTINGS

A feature which renders the machine peculiarly serviceable in special investigation is the very accessible location of the stationary test specimen. The metal is readily examined during the continuance of the test, either directly or by means of instruments fitted to the test specimen.

Furnaces, coolers or baths containing oil or chemical reagents are readily placed around the test specimen after it has been secured in position. In special tests designed to investigate chemical action in relation to fatigue, jets and sprays may be directed onto the surface of the metal.

Extensometers of the Tuckermann type can be fitted to the test specimen to measure its strain, or can be used in conjunction with the weighing bar of the machine to indicate the form of the hysteresis loop in the stress-strain diagram. The balancing ammeter, however, in conjunction with the gearing of the adjustable head of the machine affords an exceedingly sensitive extensometer that is often serviceable and is always at hand.

Thermocouples of a differential type have proved reliable and convenient for observing hysteresis during the continuance of fatigue tests. A wire zig-zag comprising some 40 junctions in series is bound to the test specimen on a thin layer of rubber tape and is connected to a recording galvanometer. The differential connections of the junctions, in two "hot" groups at mid-length in the test specimen and in two "cold" groups about 5 in. apart, is designed to insure that the galvanometer immediately indicates any generation of heat within the test specimen although it disregards any conduction of heat from end to end. A test specimen of uniform section rarely breaks if hysteresis continues to decrease for any long-continued period of time.

## TESTS ON MODELS

The field of application for which the large machine was primarily designed, and in which it is expected to yield results of immediate practical value, is the testing of models of joints and fastenings of different types used in structures and machines.

As is well known, such fastenings are designed in current practice in two distinct stages: (1) the parts are arranged and proportioned so that the calculated values of the stresses induced by specified loads are acceptable in the light of tests carried out on the materials to be used; and (2) the actual parts constructed from such designs are tried in service under working conditions and modified if and where necessary. The second stage, although exceedingly costly in time, money and life, is at present inevitable in many cases—theory having established no completely satisfactory substitute for experience.

The function of model tests, in general, is not merely to reduce in so far as may be possible the number of problems that have to be solved by experience, but to enable the designer to investigate the magnitude of risks that could not be contemplated in practice if the experiment involved the safety of a ship or train. In designing a model to be tested in a machine, the engineer is free to try all sorts of innovations; and if the model does not break, he can reduce scantlings until trouble of some sort indicates the limit of reductions. In current practice, scantlings are commonly increased where experience calls for more strength, but are seldom reduced where all goes smoothly.

Static tests, such as may be carried out in any large universal testing machine, are serviceable in many cases but unfortunately misleading in others. Under modern conditions of service, breakdowns in machinery arise more commonly from fatigue fracture than from plastic yielding. It is interesting to note that the shape of a fastening designed to give uniform fatigue strength in all sections is often quite different from that of a corresponding part designed to give uniform yield strength, and that it varies perceptibly with the choice of metal—the influence of stress-raisers being more evident in certain classes of steel than in others.

The open construction of the upper part of the Haigh machine facilitates the testing of large models. When the model is in the form of a beam, requiring falsework to hold the ends, it is preferable that this falsework be attached to the upper grip, rather than to the lower; and in any case it is desirable that the model and falsework should be dynamically in balance about the axis of loading. If the model be unduly flexible, in the sense that it deflects too far under the range of applied load, the air-gap clearance of the machine may be temporarily doubled if the range of load be restricted to one-quarter of the full working range by using series connections for the primary coils of the magnets.

In general, it is hoped that the introduction of the new machine may open up new fields of service and tend to bring fatigue testing more closely into touch with experience.

*Acknowledgment.*—In the practical development of the machine to its present successful form Mr. John D. Brunton of Musselburgh, Scotland, has supported the authors with untiring confidence and with the resources of his firm.

# FATIGUE TESTS IN SHEAR OF THREE NON-FERROUS METALS

By H. F. Moore[1] and R. E. Lewis[2]

## Synopsis

In view of the very few test data on the fatigue strength in shear for non-ferrous metals, the results of a few tests on copper, brass and duralumin are given. The testing machines and test specimens used are described. Fatigue tests were made in reversed flexure, reversed torsion, and with cycles of torsion varying from zero to a maximum.

The endurance limit in reversed torsion bears a higher ratio to the endurance limit in reversed flexure than is the case for ferrous metals, the ratio being 0.67 for copper, 0.78 for brass and 0.71 for duralumin. The values of ratio between endurance limit for zero to a maximum torsion and reversed torsion were found to be 1.90 for copper, 1.86 for brass and 1.83 for duralumin.

## Purpose of Tests

Very few test data are available on the fatigue strength in shear of non-ferrous metals. Although the tests herein reported cover only a very small portion of the field, they are tests on common non-ferrous metals, and it is believed they do offer some information, even though it be not extensive, in an unknown field. The outstanding results obtained are the relative values of endurance limit under (1) cycles of reversed flexure (tensile-compressive stress), (2) cycles under reversed torsion (shearing stress), and (3) cycles of torsional stress ranging from zero to a maximum. The tests were all made in the Fatigue of Metals laboratory of the University of Illinois.

## Materials

The materials tested were:

1. Commercially pure copper.

2. Brass rod with 62 per cent copper, 2.8 per cent lead, a slight amount of iron and the remainder zinc.

3. Duralumin designated as 17ST rod. The chemical composition was aluminum about 95 per cent, copper 4 per cent, manganese and magnesium each 0.5 per cent.

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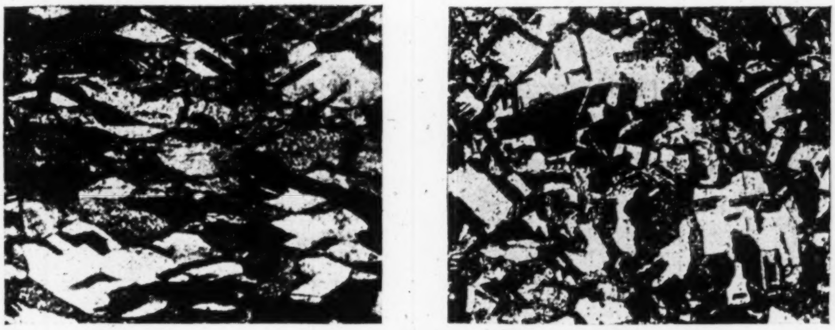
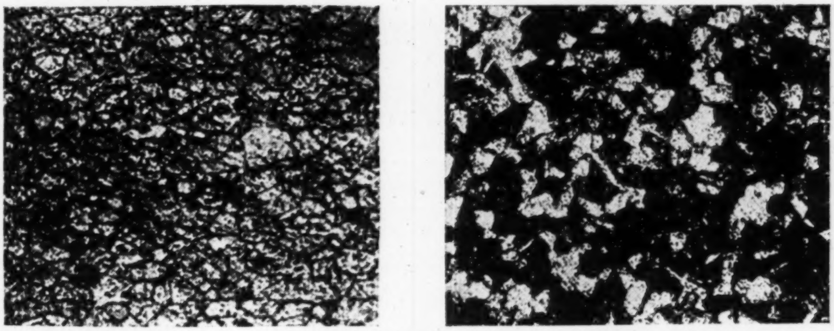
[1] Professor of Engineering Materials, University of Illinois, Urbana, Ill.

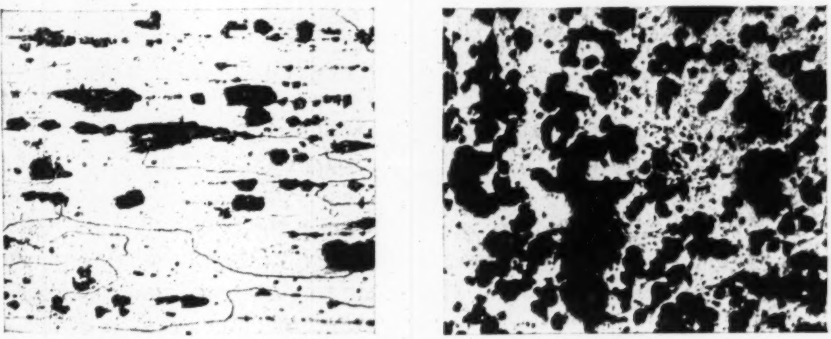
[2] Graduate Research Assistant, Mechanical Engineering, University of Illinois, Urbana, Ill.



Copper, etched with 10-per-cent ferric chloride and 30-per-cent hydrochloric acid.



Brass, etched with 10-per-cent ferric chloride and 30-per-cent hydrochloric acid.



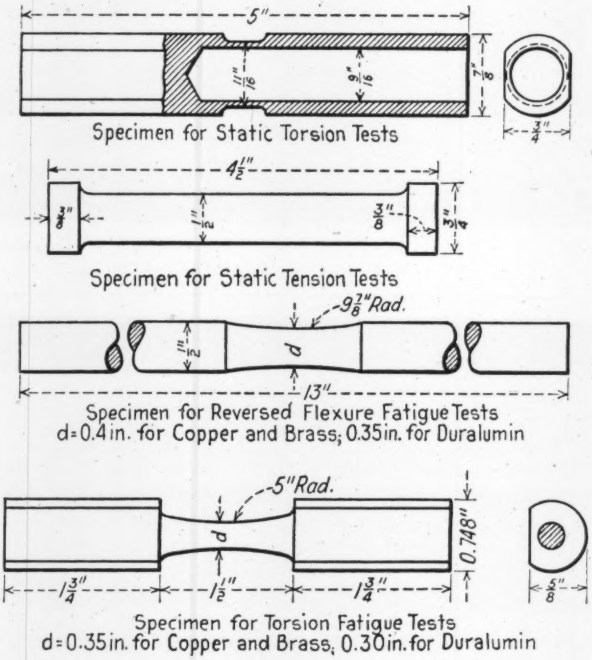
Duralumin, etched with 10-per-cent hydrofluoric acid and 15-per-cent hydrochloric acid.

FIG. 1.—Photomicrographs of Metals Tested (× 200).

All the metals were bought in the open market from a jobber and were received in the form of $\frac{3}{4}$-in. rod and tested as received. The copper rod was obviously hard drawn. Figure 1 shows micrographs of the metals tested.

## TEST SPECIMENS AND TESTING MACHINES

Figure 2 shows the test specimens used for the various tests. Attention is called to the hollow torsion specimen for the static torsion tests, and to the torsion specimen for fatigue tests. All fatigue specimens were finished with a 00 emery paper polish over the reduced section.



Specimen for Static Torsion Tests

Specimen for Static Tension Tests

Specimen for Reversed Flexure Fatigue Tests
d = 0.4 in. for Copper and Brass; 0.35 in. for Duralumin

Specimen for Torsion Fatigue Tests
d = 0.35 in. for Copper and Brass; 0.30 in. for Duralumin

FIG. 2.—Test Specimens.

The static tension tests were made on a 100,000-lb. Amsler testing machine, and stress-strain diagrams were obtained by the semi-autographic recording method. The static torsion tests were made on a Riehle 10,000-in-lb. hand power torsion machine, and a dial-gage torsion-measuring instrument was used.

The fatigue tests in reversed flexure were made on a rotating-beam machine with two symmetrically placed loads on the specimen. This machine is of the "Sondericker" or "Farmer" type—a type which is very common and has frequently been described before the Society.[1]

Figure 3 shows the machine used for the fatigue tests in torsion. It is the same in principle as the Hankins machine developed in England[2] but differs from it in detail. Torsion in the specimen *S* is set up through the jaw *A′* and the lever arm *B* by the variable throw cam *C*. The torsion is transmitted to a standard specimen *D* through jaw *A″* and the angle of twist in *D* is proportional to the twisting moment on the specimen and is measured by an indicator arm *R* and dial gages *M*. At the outer end of the standard specimen *D* is an adjusting lever *F* by which the range of stress may be varied. The two ranges used in these tests were: (1) completely reversed torsion, and (2) torsion varying from zero to a maximum. The specimen *S* is held from turning by means of broad keys bearing against flat areas on each end of the specimen. The dials *M* are held back from touching the twist-indicating rod *R* by springs. When a reading is taken the plunger of each dial in turn is pushed forward until it touches the rod *R* at one extreme of its swing. This point can be determined quite delicately by touch or by noting the vibration of the dial. The dial is read for this position. Comparisons of dial positions at running speed and while turning the machine slowly by hand gave very close agreement.

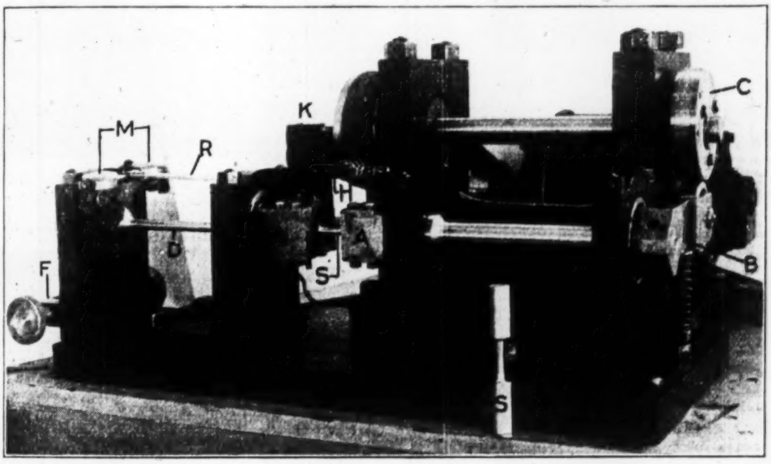


FIG. 3.—Testing Machine for Torsion Fatigue Tests.

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[1] H. F. Moore, J. B. Kommers, and T. M. Jasper, "Fatigue or Progressive Failure of Metals Under Repeated Stress," *Proceedings*, Am. Soc. Testing Mats., Vol. 22, Part II, p. 267 (1922).

[2] "Torsional Fatigue Tests on Spring Steels," *Special Report No. 9*, Department of Scientific and Industrial Research (British).

In Fig. 3, *H* represents an electrical contact which is set so that it barely misses closing while the specimen is under test. When the specimen breaks, contact is made and a relay opens a switch and stops the motor. The revolution counter *K* gives the number of cycles applied.

## TEST DATA AND RESULTS

The results of the static tests of the metals is given in Table I. Figure 4 gives the *S–N* diagrams for the fatigue tests. It will be seen that, as is usual with non-ferrous metals, the endurance limit is not so clearly marked as for iron and steel. For purposes of comparison, endurance limits have been taken at 100,000,000 cycles of stress. In some cases slight extrapolation of the diagrams was necessary.

TABLE I.—RESULTS OF STATIC TENSION AND STATIC TORSION TESTS.

| | Elastic Limit,[a] lb. per sq. in. | | Ultimate Strength, lb. per sq. in. | | Elongation in 2 in., per cent | Reduction of Area, per cent | Brinell Hardness Number |
|---|---|---|---|---|---|---|---|
| | Tension | Torsion (Shearing Stress) | Tension | Torsion (Shearing Stress) | | | |
| Copper | 41 200 | 21 400 | 48 700 | 33 300 | 14.8 | 54 | 96[b] |
| Brass | 32 000 | 29 400 | 60 400 | 42 200 | 22.8 | 47 | 124[c] |
| Duralumin | 32 000 | 20 000 | 61 000 | 45 300 | 20.5 | 37 | 118[c] |

[a] Method II, Tentative Methods of Tension Testing of Metallic Materials (E 8-27 T), *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 1067 (1927); also 1930 Book of A.S.T.M. Tentative Standards, p. 759.

[b] 500-kg. load. [c] 3000-kg. load.

TABLE II.—RESULTS OF REPEATED-STRESS (FATIGUE) TESTS.

| | Endurance Limit,[a] lb. per sq. in. | | | Endurance Limit Ratios | | |
|---|---|---|---|---|---|---|
| | Reversed Flexure | Reversed Torsion | Zero to Maximum Torsion | Reversed Flexure to Tensile Strength | Reversed Torsion to Reversed Flexure | Zero to Maximum Torsion to Reversed Torsion |
| Copper | 15 000 | 10 000 | 19 000 | 0.31 | 0.67 | 1.90 |
| Brass | 18 000 | 14 000 | 26 000 | 0.30 | 0.78 | 1.86 |
| Duralumin | 17 000 | 12 000 | 22 000 | 0.28 | 0.71 | 1.83 |

[a] For 100 million cycles of stress.

Table II gives the results of the fatigue tests. The *ratios* of the various endurance limits are perhaps of fully as much interest as the actual values obtained. The ratios of endurance limit in reversed flexure to static tensile strength are not at all unusual for the three metals tested. The ratios of the endurance limit for reversed torsion to that of reversed tension are distinctly higher than is the average for the ferrous metals. This average for ferrous metals has been found to be about 0.53, although values have been obtained ranging from 0.44 to 0.71.[1]

The ratio of endurance limit for cycles of torsional stress varying from zero to a maximum to the endurance limit for reversed torsion are about the same magnitude as the ratios observed by McAdam,[2] Moore and Jasper,[3] and Hankins.[4]

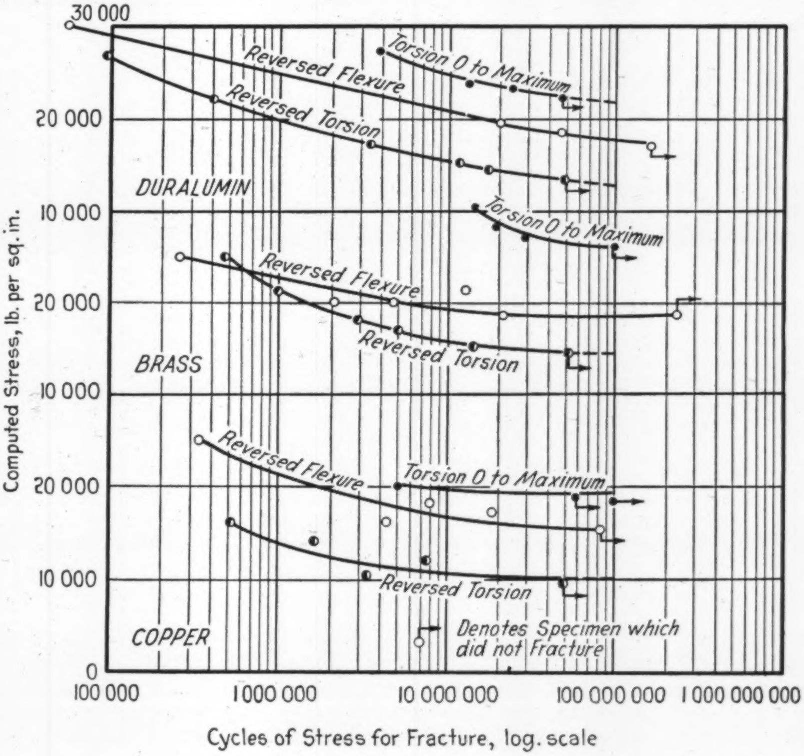

FIG. 4.—*S-N* Diagrams for Fatigue Tests of Duralumin, Brass, and Copper.

If for endurance limit under repeated torsion stress the *range* of stress were constant, whatever the ratios of the maximum or minimum values, the ratio in the last column of Table II should be 2.0. The test results indicate that as the *maximum* stress during a cycle increases, the available *range* of stress at endurance limit for these metals diminishes slightly, though much less than the corresponding diminution of range with increasing maximum stress which has been found, by other investigators, for cycles of tensile-compressive stress, and again the results are in harmony with the results obtained by McAdam, Moore and Jasper, and Hankins for tests on steel specimens.

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[1] H. F. Moore and J. B. Kommers, "Fatigue of Metals," p. 147, McGraw-Hill Book Co., Inc., New York City.

[2] D. J. McAdam, Jr., "The Endurance Range of Steel," *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 574 (1924).

[3] H. F. Moore and T. M. Jasper, *Bulletin No. 142*, University of Illinois Engineering Experiment Station.

[4] G. A. Hankins, "Torsional Fatigue Tests on Spring Steels," *Special Report No. 9*, Department of Scientific and Industrial Research (British).

[For Discussion on Fatigue of Non-Ferrous Metals, see page 257.—ED.]

# THE STATIC AND FATIGUE PROPERTIES OF BRASS

By J. B. Kommers[1]

## Synopsis

Brass bars of three different compositions were obtained in the form of cold-drawn ¾-in. rounds. These bars were tested as received, after being fully annealed, after a low-temperature anneal, and also after being fully annealed and then subjected to various amounts of cold work in tension and compression. The tests were designed to determine especially the effect of various kinds of cold work on the static and fatigue properties.

The tension tests determined the proportional elastic limit, Johnson's elastic limit, proof stress, modulus of elasticity, tensile strength and ductility. The compression tests determined the proportional elastic limit, Johnson's elastic limit, proof stress, and modulus of elasticity. The endurance limit in fatigue and the Rockwell hardness were determined.

One of the outstanding results shown by the tests was the great effectiveness of a low-temperature anneal, following cold work, in improving the static and fatigue properties. This improvement was especially remarkable for the proportional elastic limit, which in some cases was increased more than 100 per cent.

## Introduction

The purpose of this investigation was to determine the static and fatigue properties of three different brasses, having in mind especially the study of the effect on these properties of various kinds and various amounts of cold work. The experiments were carried out in the laboratories of the University of Wisconsin.

*Acknowledgment.*—The material for the tests was furnished by W. H. Bassett, Technical Superintendent and Metallurgist of the American Brass Co., Waterbury, Conn.

## Materials, Tests and Test Specimens

The materials used in the tests consisted of three different brasses, supplied in the form of ¾-in. round bars. The three brasses were marked Nos. 1, 2, and 3 and were approximately 80–20, 70–30, and 60–40 brasses. The chemical composition is given below:

| | Copper, per cent | Zinc, per cent | Lead, per cent | Iron, per cent | Tin, per cent |
|---|---|---|---|---|---|
| No. 1 (80–20)...... | 79.51 | 20.38 | 0.09 | 0.02 | .... |
| No. 2 (70–30)...... | 70.03 | 29.92 | 0.02 | 0.02 | .... |
| No. 3 (60–40)...... | 59.84 | 40.03 | 0.09 | 0.03 | 0.01 |

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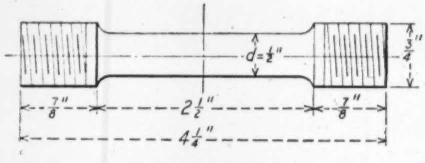
[1] Professor of Mechanics, Engineering College, University of Wisconsin, Madison, Wis.

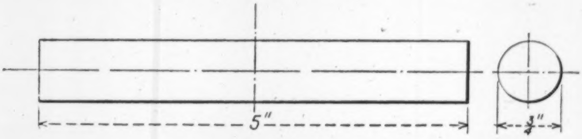
The materials had received the following treatment in fabrication:

The 80–20 and the 70–30 brasses were cast, cold rolled to $1\frac{13}{32}$ in. in diameter, annealed, cold rolled to $1\frac{7}{32}$ in. in diameter, annealed, cold rolled to $1\frac{1}{32}$ in. in diameter, annealed, cold rolled to $\frac{7}{8}$ in. in diameter, annealed and then drawn to $\frac{3}{4}$-in. diameter. The rods were then straightened in a Medart straightener.

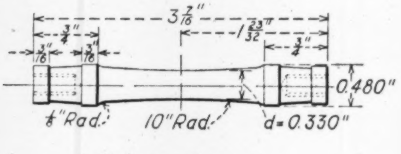
The 60–40 brass was extruded to 1-in. diameter, annealed, drawn to 0.866 in. in diameter, annealed, drawn to $\frac{3}{4}$-in. diameter, and then Medart straightened.



Tension Specimen



Compression Specimen



Fatigue Specimen

FIG. 1.—Tension, Compression and Fatigue Specimens.

In the last drawing operation, therefore, the 80–20 and the 70–30 brass rods were lengthened about 36 per cent, and the 60–40 brass rods were lengthened about 33 per cent.

The tests which were applied to the specimens were tension, compression, Rockwell hardness, and fatigue tests, the deformations in the tension and compression specimens being measured with a modified Martens' mirror apparatus. This apparatus employed a scale divided into fiftieths of an inch, and readings were taken to 0.01 in. The smallest reading represented a deformation of about 0.000021 in. The gage length usually employed was 2 in., but a few tension tests were made using an 8-in. gage length.

The tension tests were made on specimens $\frac{1}{2}$ in. in diameter, the ends being threaded and held in spherical seated holders. The compression tests were made on specimens 5 in. long and $\frac{3}{4}$ in. in diameter, a spherical bearing being used at the top. The tests were carried out on a 100,000-lb. Riehle testing machine. Figure 1 shows the tension, compression, and fatigue specimens.

In making the fatigue tests, R. R. Moore machines were employed, which have been described before the Society.[1] The specimens were tested as rotating beams, loads being applied at two symmetrical points, and the specimens were thus subjected to completely reversed stresses in tension and compression.

Some of the tension and compression tests and also some of the micrographs indicated that the 60–40 brass as received was not uniform. Some of the results in tension and compression ran higher and some lower than was expected in comparison with some of the other test results. Also, the material as received and as annealed at 527° F. showed a difference under microscopic examination which could not be accounted for unless the original material was not uniform.

## RESULTS OF TESTS

In order that the discussion may not be confusing, the results for each brass will be given separately.

### *80–20 Brass:*

Table I gives the results obtained on the 80–20 brass, each tensile and compressive result being the average of two tests. The results marked "Set Tests" were obtained from tests in which the specimen was subjected to a small initial load, and after each loading increment the load was reduced to the initial load. These tests made possible the determination of permanent sets. The values of "Proof Stress" were obtained by means of curves showing these permanent sets. The "Proof Stress" is defined as that unit stress at which the permanent unit deformation was 0.00005 in. per inch, or a unit deformation of 0.005 per cent. The purpose of the set curves was to determine the stress corresponding to a small permanent set, and beyond which the permanent set definitely and steadily increased.

A general statement may be made regarding the stress-deformation curves obtained on all three brasses both in tension and compression. There was no indication of a yield point such as is found in the case of steel, and in the opinion of the author the term yield point is misleading when used in connection with a material like brass. The stress-deformation curve for brass, both in tension and compression, was found to be a straight line for some distance along the initial curve, and then turned gradually to the right, being convex upward. In a few cases some of the completely annealed specimens and some which had been cold worked only a small amount, showed curves which became practically horizontal. Even this phenomenon, however, had been preceded by considerable permanent deformation.

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[1] R. R. Moore, "Some Fatigue Tests on Non-Ferrous Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part II, p. 66 (1925).

As shown in Table I the endurance limit of the cold-drawn material as received was 21,500 lb. per sq. in., and the tensile strength was 72,700 lb. per sq. in. The endurance ratio is therefore about 30 per cent. The proportional elastic limit is much lower than the endurance limit, and Johnson's elastic limit and the proof stress are much higher. The difference between the proportional and Johnson's elastic limit probably indicates the presence of considerable internal stress due to the cold working which the material had received. Elastic strength as measured either by the proportional elastic limit or by the proof stress is evidently not a criterion of the behavior under repeated stresses.

In the compression tests on the cold-drawn material the proportional elastic limit is not greatly different from that in tension, but Johnson's elastic limit and the proof stress are both considerably lower.

The material which was fully annealed was heated to a temperature of 1100° F., held for one hour, and then quenched in water. The endurance limit was 22,400 lb. per sq. in. and the tensile strength was 45,300 lb. per sq. in., giving an endurance ratio of about 49 per cent. The proportional elastic limit in tension for the annealed material is considerably lower than for the cold drawn, and Johnson's elastic limit and the proof stress are very much lower. In compression all three of these results are very much lower than the corresponding values for the cold-drawn material.

One of the interesting things to note is that the endurance limit of the annealed material was 900 lb. per sq. in. higher than for the cold drawn. The cold drawing had increased the tensile strength 61 per cent, but the endurance limit was actually reduced. One of the factors which produced this result was the presence of internal stresses in the cold-drawn material, but McAdam[1] has shown that another influence may be operating. He found that when the cold work increases the tensile strength beyond a certain limit, the endurance limit will no longer be increased in proportion to the increase in tensile strength.

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[1] D. J. McAdam, Jr., "Effect of Cold Working on Endurance and Other Properties of Metals, Part I," *Transactions*, Am. Soc. Steel Treating, Vol. VIII, July–December, p. 782 (1925).

This limit was found to be 50 per cent for nickel, and no less than 50 per cent for copper and constantan.

For the material annealed at 1100° F., as for the cold-drawn material, the proportional elastic limit, Johnson's elastic limit, and the proof stress are in general very much lower in compression than in tension.

The fact that internal stresses were present in the cold-drawn material is made clear by the next results shown in Table I. The cold-drawn material was heated in oil up to 527° F., held for one hour, and then cooled in air. This low temperature annealing has been shown by a number of experimenters to be very effective in relieving internal stresses. The exact temperature and time to produce the optimum effect for a given material must be determined by experiment. For the material used in these tests, other experimenters had shown that a temperature of 527° F. was at least close to that producing the optimum effect, and it was adopted for that reason. Heating the specimens in oil permitted the temperature to be controlled very closely.

TABLE I.—RESULTS OF TESTS ON 80–20 BRASS.

| Kind of Test | Treatment | Gage Length, in. | Proportional Elastic Limit, lb. per sq. in. | Johnson's Elastic Limit, lb. per sq. in. | Proof Stress, lb. per sq. in. | Modulus of Elasticity, lb. per sq. in. | Elongation, per cent | Reduction of Area, per cent | Tensile Strength, lb. per sq. in. | Rockwell Hardness, "B" Scale | Endurance Limit, lb. per sq. in. | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tension..... | Cold drawn as received.... | 2 | 14 000 | 41 200 | .... | 14 710 000 | 17.8 | 61.2 | 72 700 | 80.8 | 21 500 | .... |
| Tension..... | | 2 | 21 000 | 43 100 | 41 400 | 14 860 000 | .... | .... | .... | .... | .... | Set Test |
| Compression | | 2 | 15 900 | 28 600 | .... | 14 290 000 | .... | .... | .... | .... | .... | .... |
| Compression | | 2 | 16 400 | 29 900 | 27 400 | 15 040 000 | .... | .... | .... | .... | .... | Set Test |
| Tension..... | Annealed at 1100° F.... | 8 | 10 100 | 11 300 | .... | 15 910 000 | 50.9 | 73.2 | 45 300 | 17.3 | 22 400 | .... |
| Tension..... | | 8 | 9 500 | 11 200 | 12 200 | 16 950 000 | .... | .... | .... | .... | .... | Set Test |
| Compression | | 2 | 6 100 | 10 300 | .... | 14 600 000 | .... | .... | .... | .... | .... | .... |
| Compression | | 2 | 3 600 | 6 700 | 6 000 | 16 790 000 | .... | .... | .... | .... | .... | Set Test |
| Tension..... | Annealed at 527° F..... | 2 | 33 100 | 46 000 | .... | 15 040 000 | 21.8 | 64.4 | 70 300 | 80.5 | 25 000 | .... |
| Tension..... | | 2 | 32 900 | 45 500 | 44 000 | 15 270 000 | .... | .... | .... | .... | .... | Set Test |
| Compression | | 2 | 27 200 | 40 300 | .... | 15 050 000 | .... | .... | .... | .... | .... | .... |
| Compression | | 2 | 26 100 | 41 300 | 28 600 | 15 220 000 | .... | .... | .... | .... | .... | Set Test |
| Tension..... | Annealed at 1100° F. Shortened 20 per cent.... | 2 | 9 400 | 13 300 | .... | 16 140 000 | 51.0 | 72.4 | 48 500 | 67.2 | 25 300 | .... |
| Tension..... | | 2 | 11 500 | 24 100 | 21 800 | 16 850 000 | .... | .... | .... | .... | .... | Set Test[a] |
| Compression | | 2 | 17 300 | 33 300 | .... | 16 290 000 | .... | .... | .... | .... | .... | .... |
| Compression | | 2 | 15 200 | 35 700 | 19 400 | 14 930 000 | .... | .... | .... | .... | .... | Set Test |

[a] Annealed at 527° F. after cold working.

The endurance limit of the material annealed at 527° F. was 25,000 lb. per sq. in., and the tensile strength was 70,300 lb. per sq. in. Compared with the cold-drawn material, the tensile strength had been reduced 3 per cent and the endurance limit increased 16 per cent. Compared with the material annealed at 1100° F. the tensile strength had been increased 55 per cent and the endurance limit 12 per cent. It is seen that the increase in endurance limit is not in proportion to the increase in tensile strength. This may be due to the fact that the optimum conditions of temperature and time did not obtain in the low temperature annealing, but it is more likely that the amount of cold work was above the limit producing a proportional increase in endurance limit.

One interesting fact, which was very obvious while the extensometer tests were being made, was that the cold-drawn material annealed at 527° F. was very much more elastic than the cold-drawn material as received. The proportional elastic limit in tension was more than doubled, and in compression was almost doubled. Johnson's elastic limit and the proof stress were higher, but only moderately so. This reinforces again the conclusion that the proportional elastic limit is not at all a criterion of the strength of material under repeated stresses. In the present case the proportional elastic limit in tension was increased 136 per cent by annealing at 527° F., while the endurance limit was increased only 16 per cent. Johnson's elastic limit and the proof stress, both in tension and compression, show no consistent relation to endurance limit, and therefore cannot be used as criteria of endurance under repeated stresses.

Tests of various materials, including concrete, have shown that under repeated stresses the materials have the power to adjust themselves to a certain maximum stress which they can endure apparently indefinitely, but the elastic behavior in static tests does not help in determining the strength under cyclic stresses.

On the other hand, there does seem to be a somewhat more consistent relation between endurance limit and ultimate strength values such as tensile strength, transverse strength, compressive strength, and even Brinell hardness. In the present tests the material annealed at 1100° F. had an endurance ratio of 49 per cent, and the material annealed at 527° F. had an endurance ratio of 36 per cent. It is quite possible that the cold work here used was greater than that producing the optimum effect for combined tensile strength and endurance ratio.

The next results in Table I show the effect of taking a specimen which had been annealed at 1100° F. and shortening it from 5 to 4 in., a cold shortening of 20 per cent. As the last column in Table I shows, some of the static test specimens were annealed at 527° F. and some were not. As compared with the material annealed at 1100° F., the tensile strength was increased about 7 per cent, the endurance limit about 13 per cent, while the proportional elastic limit and Johnson's elastic limit in tension were not greatly different. In compression, on the other hand, the proportional elastic limit, Johnson's elastic limit, and the proof stress were from 3 to 5 times as large as for the material annealed at 1100° F. This action is in line with the phenomenon pointed out by Bauschinger, that overstrain in one direction greatly reduces the elastic limit for the opposite kind of stress.

*70-30 Brass:*

Table II shows the results obtained on the 70–30 brass, each tensile and compressive result being the average of two tests. The cold-drawn material had a tensile strength of 74,100 lb. per sq. in. and an endurance limit of 22,000 lb. per sq. in., the endurance ratio being 30 per cent. The proportional elastic limit is lower than the endurance limit, and Johnson's elastic limit and the proof stress are much higher, as was the case with the 80–20 brass.

TABLE II.—RESULTS OF TESTS ON 70–30 BRASS.

| Kind of Test | Treatment | Gage Length, in. | Proportional Elastic Limit, lb. per sq. in. | Johnson's Elastic Limit, lb. per sq. in. | Proof Stress, lb. per sq. in. | Modulus of Elasticity, lb. per sq. in. | Elongation, per cent | Reduction of Area, per cent | Tensile Strength, lb. per sq. in. | Rockwell Hardness, "B" Scale | Endurance Limit, lb. per sq. in. | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tension..... | Cold drawn as received.... | 2 | 17 800 | 39 700 | .... | 15 520 000 | 20.5 | 57.6 | 74 100 | 81.3 | 22 000 | .... |
| Tension..... | | 2 | 25 700 | 38 600 | 43 100 | 15 740 000 | .... | .... | .... | .... | .... | Set Test |
| Compression | | 2 | 15 800 | 28 000 | .... | 15 310 000 | .... | .... | .... | .... | .... | .... |
| Compression | | 2 | 21 500 | 29 000 | 26 500 | 14 500 000 | .... | .... | .... | .... | .... | Set Test |
| Tension..... | Annealed at 1100° F...... | 8 | 9 200 | 10 700 | .... | 14 890 000 | 55.3 | 75.1 | 47 200 | 11.2 | 22 500 | .... |
| Tension..... | | 8 | 9 700 | 12 300 | 13 900 | 15 620 000 | .... | .... | .... | .... | .... | Set Test |
| Compression | | 2 | 2 900 | 6 700 | .... | 12 230 000 | .... | .... | .... | .... | .... | .... |
| Compression | | 2 | 3 600 | 8 800 | 3 700 | 13 650 000 | .... | .... | .... | .... | .... | Set Test |
| Tension..... | Annealed at 527° F..... | 2 | 39 100 | 49 700 | .... | 15 350 000 | 23.3 | 59.8 | 73 500 | 81.5 | 28 000 | .... |
| Tension..... | | 2 | 35 400 | 49 500 | 45 500 | 15 620 000 | .... | .... | .... | .... | .... | Set Test |
| Compression | | 2 | 34 000 | 46 200 | .... | 15 400 000 | .... | .... | .... | .... | .... | .... |
| Compression | | 2 | 34 000 | 46 700 | 40 300 | 15 290 000 | .... | .... | .... | .... | .... | Set Test |
| Tension..... | Annealed at 1100° F. Shortened 20 per cent | 2 | 9 400 | 13 900 | .... | 16 270 000 | 58.3 | 70.6 | 51 300 | 62.5 | 25 000[a] | .... |
| Tension..... | | 2 | 7 900 | 14 100 | 13 500 | 16 150 000 | .... | .... | .... | .... | .... | Set Test |
| Compression | | 2 | 20 800 | 30 900 | .. | 15 980 000 | .... | .... | .... | .... | .... | .... |
| Compression | | 2 | 13 600 | 28 800 | 24 300 | 15 240 000 | .... | .... | .... | .... | .... | Set Test |

[a] Annealed at 527° F. after cold working.

In compression, as was the case with the 80–20 brass, the proportional elastic limit is not greatly different from that in tension, but Johnson's elastic limit and the proof stress are much lower.

Some of the cold-drawn material was heated to a temperature of 1100° F., held for one hour, and quenched in water. The tensile strength was 47,200 lb. per sq. in. and the endurance limit 22,500 lb. per sq. in. The cold-drawn as compared with the annealed material showed an increase in tensile strength of 57 per cent, but an actual slight decrease in endurance limit, the endurance ratio being 40 per cent. The proportional elastic limit, Johnson's elastic limit, and the proof stress are all very much lower for the annealed than for the cold-drawn material. Again it is plain that neither the elastic limits nor the proof stress can be taken as measures of the strength under cyclic stresses.

Both elastic limits and also the proof stress are very much lower in compression than in tension.

The next set of results, on material annealed at 527° F., shows again that internal stresses were present in the cold-drawn material. Compared with this material, the tensile strength was decreased about 1 per cent, and the endurance limit was increased 27 per cent, making the endurance ratio 38 per cent. Compared with the material annealed at 1100° F., the cold work and low-temperature anneal had increased the tensile strength 56 per cent, while the endurance limit was increased only 24 per cent.

TABLE III.—RESULTS OF TESTS ON 60–40 BRASS.

| Kind of Test | Treatment | Gage Length, in. | Proportional Elastic Limit, lb. per sq. in. | Johnson's Elastic Limit, lb. per sq. in. | Proof Stress, lb. per sq. in. | Modulus of Elasticity, lb. per sq. in. | Elongation, per cent | Reduction of Area, per cent | Tensile Strength, lb. per sq. in. | Rockwell Hardness, "B" Scale | Endurance Limit, lb. per sq. in. | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tension..... | Cold drawn as received.... | 2 | 17 700 | 43 900 | .... | 15 290 000 | 23.8 | 52.3 | 77 600 | 81.7 | 27 700 | .... |
| Tension..... | | 2 | 30 100 | 45 700 | 42 800 | 14 840 000 | .... | .... | .... | .... | .... | Set Test |
| Compression | | 2 | 20 400 | 34 600 | .... | 14 610 000 | .... | .... | .... | .... | .... | .... |
| Compression | | 2 | 23 100 | 35 700 | 34 500 | 15 020 000 | .... | .... | .... | .... | .... | Set Test |
| Tension..... | Annealed at 1300° F.... | 8 | 8 300 | 11 200 | .... | 13 300 000 | 40.8 | 58.2 | 59 500 | 54.2 | 21 500 | .... |
| Tension..... | | 8 | 7 700 | 11 100 | 11 200 | 14 440 000 | .... | .... | .... | .... | .... | Set Test |
| Compression | | 2 | 4 000 | 7 800 | .... | 12 880 000 | .... | .... | .... | .... | .... | .... |
| Compression | | 2 | 3 300 | 10 000 | 5 700 | 12 220 000 | .... | .... | .... | .... | .... | Set Test |
| Tension..... | Annealed at 527° F..... | 2 | 30 300 | 35 200 | .... | 15 040 000 | 31.5 | 58.2 | 73 200 | 78.9 | 30 400 | .... |
| Tension..... | | 2 | 27 800 | 35 900 | 33 300 | 14 970 000 | .... | .... | .... | .... | .... | Set Test |
| Compression | | 2 | 33 900 | 38 200 | .... | 14 720 000 | .... | .... | .... | .... | .... | .... |
| Compression | | 2 | 33 400 | 37 100 | 34 800 | 15 150 000 | .... | .... | .... | .... | .... | Set Test |
| Tension..... | Annealed at 1100° F. Shortened 20 per cent. | 2 | 9 400 | 15 200 | .... | 13 980 000 | 29.5 | 39.7 | 68 300 | 79.1 | 25 400 | .... |
| Tension..... | | 2 | 10 400 | 17 300 | 10 500 | 13 880 000 | .... | .... | .... | .... | .... | Set Test |
| Compression | | 2 | 29 300 | 55 500 | .... | 15 150 000 | .... | .... | .... | .... | .... | .... |
| Compression | | 2 | 24 900 | 52 100 | 33 000 | 14 460 000 | .... | .... | .... | .... | .... | Set Test |

[a] Annealed at 527° F. after cold working.

The proportional elastic limit, as was the case with the 80–20 brass, was more than double that of the cold-drawn material as received. In compression it was also more than double. This action of the low-temperature anneal in removing internal stresses in cold-worked material is evidently of the greatest practical importance in applications where it is imperative that the original dimensions of a device remain constant without the production of a permanent set. This is of importance, for instance, in some devices used for maintaining constant pressure or constant temperature. It is also evident that the low-temperature anneal is of considerable benefit in increasing the resistance to repeated stresses.

As was the case for the 80–20 brass, the two elastic limits and the proof stress in compression are lower than those in tension. There is also no consistent relation, for the three treatments discussed thus far, between these quantities and the endurance limit.

The next results in Table II are for material which had been annealed at 1100° F. and cold shortened 20 per cent. Only the material for the fatigue specimens was annealed at 527° F. after the cold work. Compared with the material annealed at 1100° F., the tensile strength was increased 8.5 per cent and the endurance limit 11 per cent, the endurance ratio being 49 per cent. In tension, the two elastic limits and the proof stress are not greatly different from the material annealed at 1100° F., but in compression these values are from 3 to 9 times as high. This illustrates again that overstrain in one direction is much more effective in increasing elasticity for stresses in this direction than for stresses in the opposite direction.

*60–40 Brass:*

Table III shows the results obtained on the 60–40 brass, each tensile and compressive result being the average of two tests. The cold-drawn material as received had a tensile strength of 77,600 lb. per sq. in. and an endurance limit of 27,700 lb. per sq. in., the endurance ratio being 36 per cent. As was the case for the other two brasses, the proportional elastic limit is much lower than the endurance limit, and Johnson's elastic limit and the proof stress are much higher.

In compression, as was the case for the other two brasses, the proportional elastic limit is not much different than in tension, but Johnson's elastic limit and the proof stress are much lower.

Some of the cold-drawn material was heated to a temperature of 1300° F., held for one hour, and quenched in water. The tensile strength was 59,500 and the endurance limit was 21,500 lb. per sq. in., the endurance ratio being 36 per cent. This was the only one of the three brasses in which the endurance ratio was the same for the cold-drawn and the fully annealed material, showing that the endurance limit had increased in the same proportion as the tensile strength. The two elastic limits and the proof stress are all very much lower than the corresponding results for the cold-drawn material. The cold-drawn as compared with the annealed material showed an increase in tensile strength and in endurance limit of about 30 per cent. For the other two brasses, increases in tensile strength of 61 and 57 per cent, respectively, resulted in slight reduction of the endurance limit. Apparently the 60–40 brass, consisting of two constituents, is much more susceptible to strengthening of the endurance limit by cold work than are the other two brasses consisting of one constituent.

As was the case for the other two brasses, neither the elastic limits nor the proof stress can be taken as measures of the strength under cyclic stresses.

For the 60–40 brass as for the other two brasses, in general the two elastic limits and the proof stress are considerably lower in compression than in tension.

The next set of results in Table III shows the effect of annealing the cold-drawn material at 527° F. Internal stresses in the cold-drawn material were again indicated, and compared with this material the tensile strength was reduced 6 per cent, while the endurance limit was increased 10 per cent, the endurance ratio being 42 per cent.

Compared with the material annealed at 1300° F., the cold work and low temperature anneal had increased the tensile strength 23 per cent and the endurance limit 41 per cent. It should be noted that this result is quite different from that obtained on the other two brasses, in which increases in tensile strength of 55 and 56 per cent, respectively, increased the endurance limit only 12 and 24 per cent, respectively. The 60–40 brass was the only one of the three in which the endurance limit was increased by a greater percentage than was the tensile strength.

As was the case with the other two brasses, the proportional elastic limit in tension was very much greater for the material annealed at 527° F. than for the cold-drawn material. In compression also this value was much greater.

As was the case of the other two brasses, there was no consistent relation between the two elastic limits or the proof stress and the endurance limit.

The next results in Table III are for material which had been annealed at 1300° F. and then cold shortened 20 per cent. Compared with the material annealed at 1300° F., the tensile strength had been increased 15 per cent, and the endurance limit 18 per cent, the endurance ratio being 37 per cent. This result is different from that obtained on the other two brasses in the static tests. In these tests the same amount of cold work increased the tensile strength 7 and 8.5 per cent, respectively, which is about half that obtained on the 60–40 brass. For the same amount of cold shortening, the increases in endurance limit for the other two brasses were 13 and 11 per cent, respectively, while for the 60–40 brass the increase was 18 per cent. The fatigue specimens for the 70–30 brasses were annealed at 527° F., while the 80–20 and the 60–40 specimens were not so annealed. These results indicate again that apparently the 60–40 brass is more susceptible to increase of static strength due to cold work and that the cold work also has a greater effect on the endurance limit.

As was the case with the other two brasses, a comparison of this cold-worked material with that annealed at 1300° F. shows that while the two elastic limits and the proof stress in tension are not greatly different, the compression results show values from 5 to 8 times as high.

The endurance limit was determined on material which had been cold shortened 20 per cent, both with annealing at 527° F. after the cold work, and without such annealing. The endurance limits in the two cases were practically the same. These results and similar results on the 80–20 brass indicate that 20 per cent of cold shortening does not produce much internal stress.

Table IV.—Results of Tension and Fatigue Tests.

| Material | Treatment | Proportional Elastic Limit, lb. per sq. in. | Johnson's Elastic Limit, lb. per sq. in. | Proof Stress, lb. per sq. in. | Tensile Strength, lb. per sq. in. | Endurance Limit, lb. per sq. in. | Endurance Ratio |
|---|---|---|---|---|---|---|---|
| 80-20 Brass. | Cold drawn as received......... | 14 000 | 41 200 | 41 400 | 72 700 | 21 500 | 0.30 |
| 70-30 Brass. | | 17 800 | 39 700 | 43 100 | 74 100 | 22 000 | 0.30 |
| 60-40 Brass. | | 17 700 | 43 900 | 42 800 | 77 600 | 27 700 | 0.36 |
| 80-20 Brass. | Annealed at 1100° or 1300° F......... | 10 100 | 11 300 | 12 200 | 45 300 | 22 400 | 0.49 |
| 70-30 Brass. | | 9 200 | 10 700 | 13 900 | 47 200 | 22 500 | 0.48 |
| 60-40 Brass. | | 8 300 | 11 200 | 11 200 | 59 500 | 21 500 | 0.36 |
| 80-20 Brass. | Annealed at 527° F. | 33 100 | 46 000 | 44 000 | 70 300 | 25 000 | 0.36 |
| 70-30 Brass. | | 39 100 | 49 700 | 45 500 | 73 500 | 28 000 | 0.38 |
| 60-40 Brass. | | 30 300 | 35 200 | 33 300 | 73 200 | 30 400 | 0.42 |
| 80-20 Brass. | Annealed at 1100° or 1300° F......... Shortened 20 per cent | 9 400 | 13 300 | 21 800 | 48 500 | 25 300 | 0.52 |
| 70-30 Brass. | | 9 400 | 13 900 | 13 500 | 51 300 | 25 000[a] | 0.49 |
| 60-40 Brass. | | 9 400 | 15 200 | 10 500 | 68 300 | 25 400 | 0.37 |

[a] Annealed at 527° F. after cold work.

### *Hardness Tests:*

The results of Rockwell hardness tests listed in Tables I, II, and III were determined from tests on the sides of the fatigue specimens, on the recessed portion at the ends. Each result is an average of five tests on different specimens. The results in general were fairly consistent for any one treatment, the greatest variation being found on the fully annealed 70–30 brass, in which the greatest variation was about 15 points.

The results show that the three brasses as received had practically the same hardness, and that annealing at 527° F. did not change the hardness appreciably. Fully annealing the 60–40 brass had a much smaller effect in reducing hardness than for the other two brasses.

The material which was fully annealed and then cold shortened 20 per cent had the hardness again increased, this increase being considerably greater for the 60–40 brass.

The hardness results in Tables I, II, and III show that there is no consistent relation between Rockwell hardness and the endurance limit in fatigue.

## GENERAL DISCUSSION

*Summary of Tension and Fatigue Tests:*

Table IV shows a summary of the tensile and fatigue results obtained from the three brasses with various treatments.

For the cold-drawn material as received, the 60–40 brass with the highest tensile strength has also the highest endurance limit and the highest endurance ratio. Evidently the cold work used was more suited to the 60–40 brass than to the other two. The two elastic limits and the proof stress are not greatly different for the three brasses, except that the proportional elastic limit of the 80–20 brass is low.

For the completely annealed material, the reduction in tensile strength compared with the cold-drawn material is about 36 per cent for the 80–20 and 70–30 brasses, but only 23 per cent for the 60–40 brass; and this in spite of the fact that the annealing temperature was higher for the latter brass. The result is that the endurance ratio is lower for the 60–40 brass than for the other two. The annealing has evidently greatly improved the endurance ratio of the 80–20 and the 70–30 brasses. This would tend to indicate that the cold work used in fabricating the cold-drawn material had been too drastic to obtain the best results for endurance under repeated stresses. The two elastic limits and the proof stress are not greatly different for the three brasses.

The cold-drawn material annealed at 527° F. shows improvement in the endurance ratio for all three brasses, and a quite remarkable improvement for all three brasses in proportional elastic limit.

The material which had been fully annealed and then cold shortened 20 per cent shows a better endurance ratio for the 80–20 and the 70–30 brasses than for the 60–40 brass. It may be that this milder cold work is relatively more effective for these brasses than the original cold drawing which had increased the length about 36 per cent.

The effect of the cold work on the endurance limit of the three brasses may perhaps be best judged by comparing the results of the fully annealed material with that annealed at 527° F. The absolute amounts of increase of endurance limit due to the cold work and the low-temperature anneal were 2600, 5500, and 8900 lb. per sq. in. for the 80–20, 70–30, and 60–40 brasses, respectively, the corresponding percentages being 12, 24, and 41 per cent, respectively.

*Summary of Compression Tests:*

Table V shows a summary of the compressive results obtained on the three brasses with various treatments.

For the cold-drawn material as received the corresponding results for the 80–20 and 70–30 brasses are much the same, but the results for the 60–40 brass are appreciably higher.

For the fully annealed material the two elastic limits and the proof stress vary considerably in corresponding results, and these are all much lower than those for the cold-drawn material.

TABLE V.—RESULTS OF COMPRESSION TESTS.

| Material | Treatment | Proportional Elastic Limit, lb. per sq. in. | Johnson's Elastic Limit, lb. per sq. in. | Proof Stress, lb. per sq. in. |
|---|---|---|---|---|
| 80-20 Brass. | Cold drawn as received | 15 900 | 28 600 | 27 400 |
| 70-30 Brass. | | 15 800 | 28 000 | 26 500 |
| 60-40 Brass. | | 20 400 | 34 600 | 34 500 |
| 80-20 Brass. | Annealed at 1100 or 1300 | 6 100 | 10 300 | 6 000 |
| 70-30 Brass. | | 2 900 | 6 700 | 3 700 |
| 60-40 Brass. | | 4 000 | 7 800 | 5 700 |
| 80-20 Brass. | Annealed at 527° F | 27 200 | 40 300 | 28 600 |
| 70-30 Brass. | | 34 000 | 46 200 | 40 300 |
| 60-30 Brass. | | 33 900 | 38 200 | 34 800 |
| 80-20 Brass. | Annealed at 1100° or 1300° F<br>Shortened 20 per cent | 17 300 | 33 300 | 19 400 |
| 70-30 Brass. | | 20 800 | 30 900 | 24 300 |
| 60-40 Brass. | | 29 300 | 55 500 | 33 000 |

For the cold-drawn material annealed at 527° F., the most noteworthy results are the high values of proportional elastic limit compared with those of the cold-drawn material. The values for Johnson's elastic limit and the proof stress are also higher, but not in such great proportion. The 70–30 brass shows the most consistently high increases in all the values listed. These results indicate again the marked improvement in static results due to the low-temperature anneal.

The results on the material fully annealed and then cold shortened 20 per cent may be compared with results for the fully annealed material. Corresponding results are from 3 to 7 times as high, showing that the cold work has greatly improved the static compressive properties.

## CONCLUSIONS

The general conclusions which may be drawn from these experiments are as follows:

1. All three brasses are capable of having their static strength properties greatly increased by means of various kinds of cold work.

2. All three brasses are capable of having their fatigue properties considerably increased by means of various kinds of cold work.

3. The most effective results are obtained from cold work when the cold work is followed by a low-temperature anneal. The low-temperature anneal is especially effective in increasing the proportional elastic limit.

4. When the cold work is carried out beyond a certain limit, the increase in endurance limit is in smaller proportion than the increase in tensile strength.

5. Cold work in one direction, for example in tension, will increase the elastic properties for stresses in that direction more than for stresses in the opposite direction.

6. Comparing the cold-drawn material which was annealed at 527° F. with the fully annealed material, the 60–40 brass, which was the least homogeneous, was the only one in which the increase in endurance limit was proportionately greater than the increase in tensile strength.

7. Neither the proportional elastic limit, Johnson's elastic limit, the proof stress, nor the Rockwell hardness showed a consistent relation to the endurance limit, and therefore these results cannot be used as criteria of fatigue strength. Even the tensile strength cannot be relied upon as a definite criterion of fatigue strength, the endurance ratios obtained varying from 30 to 52 per cent.

# DISCUSSION ON FATIGUE OF NON-FERROUS METALS

Mr. W. H. Bassett.[1]—I should like to call attention to the difference in the Muntz metal used in these two investigations. In the case of the metal used by Messrs. Moore and Lewis, there was present 2.8 per cent of lead and the copper was 62 per cent; in other words, they had tested what is usually known as a free turning rod. Of course it is of the Muntz metal type, but the presence of lead very considerably changes the character of the material. In other words, the material is cold short, the lead acting very much the same as shortening in bread, making the material relatively easily sheared and with low elongation, whereas the rod used by Mr. Kommers contained a very small percentage of lead, less than 0.1 per cent, and was true Muntz metal. I have not critically checked the difference in test results between these two rods, but I am sure that the lead would make quite a profound difference in the properties of the material.

Mr. J. B. Kommers[2] (*by letter*).—I wish to call attention to the fact that in Tables I, II, and III of my paper the increase of elastic strength for the cold-drawn material which had been annealed at 527° F. would be different if based upon Johnson's elastic limit instead of the proportional elastic limit. The proportional limit indicated the unit stress at which there was the first deviation from the straight line. Upon consulting the original curves I find that at Johnson's elastic limit there was a deviation from the straight line of the order of 0.0003 in. per inch. This is a relatively large amount.

In this connection attention should be called to the results called "Proof Stress," listed in Tables I, II, and III. The proof stress was obtained by measuring the permanent set after each increment of load was applied. It was the unit stress at which a permanent set of 0.00005 in. per inch was obtained. The proof stress was a unit stress corresponding to a very small permanent set, and beyond which the permanent set definitely and steadily increased. The proof stress corresponding to a permanent set of 0.00005 was about one-sixth of the deviation from the straight line corresponding to Johnson's elastic limit. In other words, the proof stress was a much more delicate measure of elasticity.

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[1] Metallurgical Manager, American Brass Co., Waterbury, Conn.

[2] Professor of Mechanics, Engineering College, University of Wisconsin, Madison, Wis.

The tables show that the proof stress for the cold-drawn material was about equal to Johnson's elastic limit and therefore much higher than the proportional elastic limit. The explanation for this behavior is no doubt the presence of internal stresses. These internal stresses caused a low proportional elastic limit, but their effect was largely overcome in connection with the proof stress by the fact that in determining the proof stress the load was reduced almost to zero after each increment of load. This repeated loading, even though it amounted to only 10 or 15 repetitions, evidently had the effect of improving the elasticity, so that the proof stress was much higher than the proportional elastic limit. Annealing the cold-drawn material at 527° F. evidently had a similar effect and also greatly improved the elasticity as measured by the proportional elastic limit.

The effect of repeated stresses in improving elasticity has been definitely shown in such a material as concrete, in which the first loading produces a stress-deformation curve which is convex upward. Repeated stressing, when the stress is within the endurance limit, will finally effect a straight-line relation between stress and strain.

# INFLUENCE OF WATER COMPOSITION ON STRESS CORROSION[1]

By D. J. McAdam, Jr.[2]

## Synopsis

Part I gives a brief résumé of previous work, an outline of continued investigation and a description of methods and material.

Part II presents new composite diagrams representing corrosion of ordinary steels in calcium carbonate water. These diagrams confirm previous conclusions that the influence of stress range and cycle frequency on corrosion pitting of all these steels is practically the same.

Part III discusses corrosion of steel in distilled water. The influence of stress range and cycle frequency on corrosion of steels in distilled water is illustrated by diagrams.

Part IV compares the effects of stress corrosion of steel in distilled water and in carbonate water. Comparison is made by means of superimposed diagrams. At high frequency, the rate of net damage is greater in carbonate water than in distilled water. At low frequency, especially at high stress, the reverse is true. Evidence indicates that total damage at low frequency and high stress eventually would be greater in distilled water than in carbonate water.

Part V is a brief general discussion of stress corrosion. Comparison is made between diagrams for monel metal and ordinary steels. Evidence indicates that the less generally corrodible of two metals in the same water (at low frequency and high stress) may receive the greater damage. Other subjects discussed are: intercrystalline corrosion, and "determining factors" in stress corrosion.

## PART I—OUTLINE OF INVESTIGATION, DESCRIPTION OF METHODS AND MATERIAL

### Description of Previous Investigation at Annapolis

Results of investigation of stress corrosion have been presented by the author in fourteen previous papers**(1-14)**.[3] In these papers reference was made to a paper by Haigh**(15)** in 1917, in which he showed that metals may fail at lower stresses if corrosion and stress are simultaneous than if the corrosion is prior to the stress.

The first part of the investigation at Annapolis dealt with the conditions necessary to cause complete failure under corrosion and cyclic stress of high frequency. The entire process was called "corrosion-fatigue." As a result of this part of the investigation, which was described in five papers(1-5), the conclusion was reached that corrosion-fatigue is due to the accelerating influence of cyclic stress on corrosion. Pits formed under the influence of cyclic stress and corrosion were found to be sharper and deeper than pits formed in the same time under stressless corrosion. These conclusions led to a general investigation of the influence of stress on corrosion. Results of the investigation have been reported in nine papers(6-14).

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[1] The experiments described in this paper were made at the U. S. Naval Engineering Experiment Station, Annapolis, Md. The paper was written after the transfer of the author to the U. S. Bureau of Standards, Washington, D. C.

[2] Metallurgist, U. S. Bureau of Standards, Washington, D. C.

[3] The boldface numbers in parentheses refer to the papers given in the list of references appended to this paper, see p. 272.

The basic process of corrosion-fatigue is corrosion differing only in degree from ordinary stressless corrosion. "Corrosion-fatigue," therefore, is not a suitable name for this basic process. The name "fatigue corrosion"(11, 12) has been used to designate corrosion under the influence of cyclic stress. This name, however, does not imply that the basic process is related to ordinary fatigue. In recent papers (12, 13, 14) consideration has been given to the influence of steady stress. The name "stress corrosion" has been used(14) to designate corrosion under the influence of stress, either cyclic or steady.

The various factors involved in stress corrosion have been discussed in previous papers(8-14) and the inter-relationship between these factors has been illustrated by diagrams of various types. The most recent paper(14) gives a general discussion of the data presented in preceding papers, with the addition of supplementary data.

In previous papers, chief attention has been given to the behavior of a great variety of metals in contact with either calcium carbonate water or Severn River water. Because of the wide range of corrosion resistance of the metals, however, the experiments made with the two kinds of water represent a wide range of corrosion intensity. In recent papers(12, 13) some attention has been given to the influence of varying water composition.

## OUTLINE OF CONTINUED INVESTIGATION

The present paper gives results of further investigation of the influence of water composition. Comparison is made between the behavior of steels in carbonate water and in distilled water. For this purpose, new composite diagrams have been made to represent stress corrosion of carbon steels and ordinary alloy steels in carbonate water. These composite diagrams are based on experiments described in previous papers(8-14) and on some more recent experiments. Stress corrosion of steel in distilled water has been investigated and the results are presented in the form of diagrams.

*Method of Investigation.*—As described in previous papers(6-14), each experiment consisted of two stages. In the first stage the specimen (stressed or unstressed) was corroded in contact with water. In the second stage, the previously corroded specimen was subjected to fatigue test in air. The fatigue limit thus obtained was then compared with the endurance limit. The lowering of the fatigue limit was used as a measure of the "damage" due to the corrosion.

*Description of Waters.*—The compositions of the waters used in the investigation are given in Table I. As the corrosion of rotating-cantilever specimens in a stream of water required a large amount of water, it was impossible to obtain a sufficient quantity of distilled water of the best grade. The distilled water used was obtained from the condenser of a steam boiler. As shown in Table I, the chlorine percentage and total solids are greater than those of distilled water of laboratory grade.

TABLE I.—COMPOSITION OF WATER, PARTS PER MILLION.

| Water | Date | Solids | Organic and Volatile Matter, O + V | Alkalinity[a] | Cl | $SO_3$ | CaO | MgO | $Al_2O_3 + Fe_2O_3$ | $SiO_2$ |
|---|---|---|---|---|---|---|---|---|---|---|
| Carbonate Water...... | April 12, 1928..... | 579.4 | 158.4 | 111.6 | 244.5 | 9.14 | 151.1 | 25.5 | none | 15.4 |
| | April 20, 1928..... | 614.0 | 108.3 | 97.4 | 240.8 | 10.7 | 70.3 | 67.8 | none | 9.8 |
| | May 22, 1928...... | 556.2 | 98.4 | 104.9 | 209.0 | 48.1 | 148.3 | 18.6 | none | 9.6 |
| | September 5, 1929. | 410.5 | 142.7 | 98.0 | 117.0 | 4.3 | 51.8 | 19.3 | none | 3.9 |
| Condenser Water...... | August 15, 1930... | 23.2 | ..... | 20.4 | 4.5 | none | none | 2.2 | 5.8 | none |
| | August 31, 1930... | 27.7 | ..... | 16.6 | 5.0 | none | none | 1.3 | 4.6 | 1.1 |
| | May 25, 1931...... | 9.5 | ..... | 5.5 | 2.9 | none | none | trace | none | none |

[a] Alkalinity is expressed in terms of $CaCO_3$.

The condenser water was allowed to cool in a large steel tank. Air was then bubbled into the water through perforated pipes. The air supply was sufficient to cause violent agitation of the water for at least an hour.

The calcium carbonate water used in experiments described in this and previous papers (1-14) was obtained from a well. On account of the drought in the summer and fall of 1930, it was necessary at times to supplement this water supply by adding a small proportion of artesian well water from the Naval Academy. The composition of the composite water, however, differed little from the normal composition of the well water.

The waters were pumped to elevated tanks in the building. From these tanks, they flowed through iron pipes and were applied in small streams to the rotating specimens. It had been thought that both waters as applied to the specimens would be practically saturated with dissolved oxygen. The distilled water proved to be practically saturated, but the carbonate water was not completely saturated.

*Metals.*—For most of the experiments to be described, a 3½-percent nickel steel, LQ–W–10, was used. This steel is of nearly the same composition, and received the same heat treatment, as the nickel steel IW–W–10 that has been used in many experiments described in previous papers(9-14). Steel LQ–W–10 had the following physical properties:

| | |
|---|---|
| Tensile strength, lb. per sq. in | 136 000 |
| Johnson's limit, lb. per sq. in | 123 200 |
| Proof stress, lb. per sq. in | 124 600 |
| Elastic limit, lb. per sq. in | 122 500 |
| Proportional limit, lb. per sq. in | 107 000 |
| Elongation in 2 in., per cent | 19.3 |
| Reduction of area, per cent | 60.4 |
| Endurance limit (estimated), lb. per sq. in | 69 000 |
| Charpy impact, ft-lb | 29.2 |

Results of tension tests are averages of eight determinations. This steel, as will be noted, is slightly stronger than steel IW–W–10.

*Diagrams.*—As in recent papers (12-14), results of the investigation have been presented in diagrams of three types. General descriptions of these types have been given in previous papers(8-14).

## PART II—COMPOSITE DIAGRAMS, CARBON STEEL AND ORDINARY ALLOY STEELS, CARBONATE WATER

*New Composite Diagrams.*—Composite diagrams representing stress corrosion of carbon steel and ordinary alloy steels in carbonate water were presented in a previous paper(9). Since then, however, experiments have been made with additional steels, using wider ranges of corrosion stress and cycle frequency. These experiments have made it possible to construct composite diagrams that are more accurate and more extensive than those presented in the previous paper(9).

*Steels Included in Composite Diagrams.*—The steels included in the new composite diagrams cover a wide range of chemical composition and physical properties. In the following list, each number in parenthesis is the reference number of the paper in which the individual diagram was presented.

| | | |
|---|---|---|
| Carbon steel | JR – W – 10 | (9) |
| Nickel steel | IW – W – 10 | (12) |
| Nickel steel | IW – 14.5 | (8) |
| Nickel steel | LQ – W – 10 | (present paper) |
| Chromium-nickel steel | BC – W – 10 | (8) |
| Chromium-nickel steel | AY – W – 10 | (8) |
| Chromium-nickel steel | AX – W – 10 | (11) |
| Chromium-nickel steel | BW – W – 10 | (13) |
| Copper steel | JK – W – 9 | (9) |

*General Description of Composite Diagrams.*—The composite diagrams are "constant net-damage" diagrams. Two diagrams have been constructed, one of each of the two types (Types 10 and 11 (*a*)) that have been discussed in previous papers. Each Type 11 (*a*) graph was first drawn separately on the basis of results of individual experiments with the various steels. The constant-frequency graphs thus drawn were then assembled in a Type 11 (*a*) diagram. From this Type 11 (*a*) diagram, the corresponding Type 10 diagram was then derived. As shown in previous papers, the Type 11 (*a*) and Type 10 diagrams illustrate the influence of stress range and cycle frequency, respectively.

## Influence of Stress Range on Corrosion of Ordinary Steels in Carbonate Water

Type 11 (*a*) graphs, each representing some one cycle frequency, are shown in the upper two divisions of Fig. 3. In each division, the graphs have the same scale of ordinates, but each graph has its own scale of abscissas, which is indicated by numbers adjacent to it. Each graph represents the stresses and times necessary to cause 15 per cent net damage at the indicated cycle frequency. Abscissas measured from right to left represent corrosion times. Measured in the opposite direction, abscissas represent rates of net damage.

In view of the wide range of composition and physical properties of the steels represented by these graphs, the scatter of experimental points is surprisingly small. The relationship between stress range and rate of net damage as shown in previous papers (10-14), is approximately a straight-line logarithmic relationship. The composite diagram also confirms the conclusion, expressed in previous papers (8-14), that the stress-rate relationship for all ordinary steels can be represented by practically the same diagram. The influence of stress on corrosion pitting, therefore, is practically the same for all carbon steels and ordinary alloy steels.

The slope of the graph representing 1.5 r.p.m., as drawn in Fig. 3 on the basis of the more extensive data, is greater than in the composite diagram presented in a previous paper (9). The graphs representing 0.5 and 1.5 r.p.m. evidently have about the same slope. The graph representing 5 cycles per hour has not enough experimental basis to establish definitely the position and slope. The slope of this graph as drawn, however, is slightly less than the slope of the graph representing 0.5 r.p.m. As illustrated by the assembled composite diagram in Fig. 3, the slope of the Type 11 (*a*) graph for steels evidently decreases slightly with decrease in cycle frequency.

The straight-line relationship indicated by these graphs may be represented by the following equation, which has been discussed in previous papers:

$$R = CS^n$$

In this equation, $R$ represents (average) rate of net damage, $S$ represents corrosion stress, and $n$ represents the cotangent of the angle of slope of the graph. The factor $C$ depends on the intensity of corrosion and on the cycle frequency. For steels in carbonate water with free access of air, as illustrated by the composite diagram, $n$ ranges from about 2.8 to 3.5.

## INFLUENCE OF CYCLE FREQUENCY ON CORROSION OF ORDINARY STEELS IN CARBONATE WATER

From the Type 11 (*a*) composite diagram shown in Fig. 3, a Type 10 diagram has been derived. It is shown, in Fig. 4, as a continuous-line diagram. The Type 10 diagram(11-14) illustrates the relationship between total number of cycles and the corrosion time necessary to cause constant net-damage. As the constant-frequency lines in this diagram are parallel, the diagram illustrates also the influence of cycle frequency on the rate of net damage.

The Type 10 diagram may be studied as a plan view of a three-dimensional diagram, of which the Type 11 (*a*) diagram is a view in the direction of the arrow in Fig. 4. This and other Type 10 diagrams, however, will be discussed as two-dimensional diagrams. In the discussion, therefore, the words "vertical," "upward," "downward," etc., refer to the direction of ordinates.

The composite graphs for steels in carbonate water, in Fig. 4, have been extended only to the limits corresponding to results of experiments. These graphs, as they extend to the right, approach a horizontal direction. On the basis of experiment, there is no clear evidence that the graphs, if extended further, would reverse their curvature. Evidence based on damage or failure of boilers(12), however, indicates that under boiler conditions, the Type 10 graphs reverse their curvature and at very low cycle frequency approach a vertical descent. One of these failed boilers used the same carbonate water that has been used in the experimental investigation. In graphs representing carbonate water at room temperature and with free access of air, however, there may be no such reversal of curvature, or the reversal may be less abrupt than in graphs representing boiler conditions. This subject will be discussed further in Part IV.

## PART III—CORROSION OF STEEL IN DISTILLED WATER

### Stressless Corrosion of Steel in Carbonate Water and in Distilled Water

The progress of corrosion of metals is well illustrated by means of Type 5 diagrams(8-14). A Type 5 graph shows the relationship between corrosion time and the fatigue limit of the previously corroded specimen, corrosion stress and cycle frequency being held constant. A Type 5 diagram consists of a series of graphs, representing the same cycle frequency but different corrosion stresses. The uppermost graph of the diagram represents stressless corrosion.

Type 5 diagrams for steels in carbonate water and in distilled water are shown in Fig. 2. In diagrams VI and VII of this figure, curves *A* and *B* represent corrosion of nickel steel LQ–W–10 in distilled water and in carbonate water respectively. Curve *C* of diagram VI represents stressless corrosion of nickel steel IW–W–10 in carbonate water. A stressless-corrosion graph for this steel, presented in a previous paper(14), was extended to about 300 days. The fact that curve B is above curve *C* is due to the fact that steel LQ–W–10 is slightly stronger than steel IW–W–10.

Curve *A*, on the basis of experiment, has been extended to about 183 days. Curve *B*, representing corrosion of the same steel in carbonate water, could not be extended beyond about 12 days on the basis of experiment alone. This curve, however, has been extended (in diagram VII) so as to have the same form as the graph previously presented(14) for steel IW–W–10. In accordance with its higher origin, curve *B* is higher than the curve for steel IW–W–10, throughout its entire length.

The initial rate of damage under stressless corrosion, as indicated by the slopes of curves *A* and *B*, is about ten times as great in carbonate water as in distilled water. Decrease in rate with time, however, is more rapid in carbonate water than in distilled water. For this reason, curves *A* and *B*, though they diverge from the origin, converge as they approach a nearly horizontal direction, with curve *A* considerably above curve *B*. Whether or not curve *A*, if extended far enough, would turn downward has not been determined.

### Stress Corrosion of Steel in Distilled Water, as Illustrated by Type 5 Diagrams

Type 5 stress-corrosion graphs for steels in carbonate water and in distilled water are qualitatively similar. The total damage due to stress corrosion is measured by the vertical distance of the stress corrosion graph (at any point) below the origin. The net damage due to the stress, however, is measured by the vertical distance of the stress-corrosion graph below the stressless-corrosion graph. The influence of stress on corrosion is measured by net damage rather than by total damage.

Quantitative investigation of stress corrosion involves a study of the influence of stress, time, and number of cycles on net damage. This subject is best studied by derived diagrams representing constant net damage. To derive such a diagram from a Type 5 diagram, a line may be drawn below the stressless-corrosion graph so that the ordinates of the line are a constant percentage of the ordinates of the stressless-corrosion graph. A line so drawn represents constant net damage. The broken lines in Figs. 1 and 2 are drawn to represent 15 per cent net damage. The coordinates of the intersections of these lines with the stress-corrosion graphs are used in constructing graphs to represent constant net damage.

## INFLUENCE OF CYCLE FREQUENCY ON STRESS CORROSION OF STEEL IN DISTILLED WATER

A Type 10 diagram is a constant net-damage diagram illustrating the influence of cycle frequency on net damage. A Type 10 diagram for nickel steel in distilled water is shown in Fig. 5. As the graphs extend to the right, they first curve so as to approach a horizontal direction, as do the graphs in Fig. 4 for steels in carbonate water. With further extension, however, the graphs for steel in distilled water reverse their curvature and turn downward. In the high-stress graphs, the reversal of curvature is at a frequency of more than 50 r.p.m. and the downward curvature is very abrupt. With decrease in corrosion stress, the reversal of curvature of the corresponding graph is continually less abrupt, and probably is at continually lower cycle frequency. Graphs representing stresses of 20,000 lb. per sq. in. or less have not been established experimentally, but have been drawn so as to represent the most probable relationship with the Type 11 (*a*) diagram to be described later.

The lower the corrosion stress, the nearer does the form of the graph for steel in distilled water approach the form of the graphs for steels, aluminum alloys (10-12), and nickel(13) in carbonate water. The high-stress graphs for steel in distilled water resemble in form the graphs for monel metal(12) and aluminum bronze(13,14) in carbonate water. With decrease in corrosion stress, therefore, there is a gradual transition in form of the Type 10 graph for steel in distilled water.

The lower, nearly vertical portions of the graphs in Fig. 5 terminate at an abscissa representing 0.5 cycle. These portions of the graphs, as shown in previous papers(12-14), give some idea of the relative influence of steady stress and cyclic stress on corrosion.

## Influence of Stress Range on Corrosion of Steel in Distilled Water

The relationship between stress range and the rate of net damage for steel in distilled water is illustrated by a Type 11 (*a*) diagram in Fig. 3. This diagram differs considerably from the diagram for steel in carbonate water. The diagram for steel in distilled water, with the exception of the lower left side, resembles the diagram for a non-ferrous metal, such as monel metal(12) or aluminum bronze(13), in carbonate water. At intermediate stresses and frequencies, the graphs for steel in distilled water are nearly straight and nearly parallel. For this part of the diagram, *n* is about 2.8 to 3.0, the same as for steels in carbonate water. The high-frequency graphs, as they ascend, curve to the right, as do the graphs for a non-ferrous metal(12-14).

The low-frequency graphs, as they descend, diverge. This divergence and the resultant increase in breadth of the diagram correspond to the great divergence of the graphs in the Type 10 diagram in Fig. 5. In the 5 revolutions per hour graph in Fig. 3, *n* ranges from about 3.5 to at least 6. In the 1 revolution per hour graph, *n* probably ranges from about 3.5 to more than 7. The graph in Fig. 3 *n* representing steady stress has been drawn to correspond to the intersections of the Type 10 graphs in Fig. 4 with the horizontal line representing 0.5 cycle. The Type 11 (*a*) graph, as drawn in Fig. 3, indicates that the rate of net damage probably varies as the fourth to at least the tenth power of the steady stress.

## PART IV—COMPARISON OF EFFECTS OF STRESS CORROSION IN DISTILLED WATER AND IN CARBONATE WATER

### Comparison of Net Damage in Distilled Water and in Carbonate Water

Type 10 diagrams for steels in distilled water and in carbonate water are superimposed in Fig. 4. At the high-frequency end, the graphs for distilled water are above the corresponding graphs for carbonate water. As they extend to the right, the graphs for both kinds of water curve toward the horizontal. The graphs for carbonate water, at least within the field based on results of experiment, continue this approach. The graphs for distilled water, however, eventually curve downward and cross the corresponding graphs for carbonate water. At high frequencies, therefore, the rate of net damage is greater in carbonate water than in distilled water. At low frequencies, especially at high stresses, the rate of net damage is greater in distilled water.

Type 11 (*a*) diagrams for steels in distilled water and in carbonate water have been superimposed in the third division of Fig. 3. To avoid confusion, two of the graphs shown in the lower division of Fig. 3 have not been included in the superimposed diagrams. The range of frequencies represented in the two superimposed diagrams, however, is the same.

At intermediate stresses, the Type 11 (*a*) diagram for distilled water is much narrower than the diagram for carbonate water. At high frequencies, evidently, the rate of net damage is greater in carbonate water than in distilled water; at low frequencies the reverse is true. This fact has been mentioned previously in discussing the superimposed Type 10 diagrams in Fig. 4. With decrease in stress, the type 11 (*a*) diagram for distilled water widens and, at low stresses, may be about as wide as the diagram for carbonate water. This increase in width of the Type 11 (*a*) diagram corresponds to the change in form of the Type 10 graph (in Fig. 5) with decrease in stress.

## THE LOW-FREQUENCY END OF THE TYPE 10 DIAGRAM FOR STEELS IN CARBONATE WATER

The Type 10 graphs for nickel steel IW–W–10, presented in a previous paper(12), were similar in form and position to the composite graphs in Fig. 4, except that the graphs in the previous paper were further extended by broken lines. These extension lines were drawn with abrupt reversal of curvature and approach to a vertical descent. The reversal of curvature and steep descent were based on numerous examples of damage or failure of steam boilers in service. The water used in one of the boilers, which exploded, was the carbonate water that has been used in the experimental investigation.

Under some conditions of corrosion, therefore, Type 10 graphs for ordinary steels in carbonate water may resemble in form the high-stress graphs for steels in distilled water. Under boiler conditions, the Type 10 graphs may possibly diverge as do the graphs for steels in distilled water. In carbonate water at room temperature and with free access of air, the Type 10 graphs may continue their approach to the horizontal direction. Or, if sufficiently extended, they may reverse their curvature gradually as do the low-stress graphs for steel in distilled water.

The composite diagram for steels in carbonate water, therefore, may represent one extreme, and the diagrams for monel metal and aluminum bronze may represent the other extreme of variation in the form of the Type 10 graph. The diagram for steels in distilled water evidently represents a transition between the two extremes.

## Total Damage of Steel in Distilled Water and in Carbonate Water

The fact that the rate of net damage at low cycle frequency is greater in distilled water than in carbonate water does not necessarily mean that ***total damage*** eventually would be greater in distilled water than in carbonate water. Total damage is equal to net damage plus the damage due to stressless corrosion. As stressless-corrosion damage is considerably less in distilled water than in carbonate water, the question arises whether this difference eventually would be overcome by the greater rate of net damage in distilled water. Although this question has not been thoroughly investigated, there is some evidence that the total damage eventually would be greater in distilled water than in carbonate water.

In diagram VI of Fig. 2 are shown two groups of stress-corrosion graphs. The continuous-line graphs are experimentally established graphs, transferred from diagram IV of Fig. 2, representing stress corrosion of steel in distilled water. The broken-line graphs represent stress corrosion of steels in carbonate water. These broken-line graphs, have not been established directly by experiment but have been drawn to correspond with the composite Type 11 (*a*) and Type 10 diagrams in Figs. 3 and 4.

As indicated by the two groups of stress-corrosion graphs (in diagram VII of Fig. 2) some of the graphs representing distilled water evidently would cross the corresponding graphs representing carbonate water. The evidence, therefore, seems to indicate that, at low cycle frequency, total damage eventually may be greater in distilled water than in carbonate water. This means that under such conditions steels would fail at lower stress or in shorter time in distilled water than in carbonate water.

## PART V—GENERAL DISCUSSION OF STRESS CORROSION

### Comparison of Type 10 Diagrams for Monel Metal and Steel in Carbonate Water

It has been shown that at low cycle frequency the less generally corrosive of two waters may cause the greater damage. This fact suggests that, of two metals under stress corrosion at low cycle frequency in the same water, the less generally corrodible may receive the greater damage. An example of this relationship is shown in Fig. 6, in which a Type 10 diagram for monel metal(12) and the composite diagram for steels have been superimposed. Both diagrams represent stress corrosion in carbonate water.

The graphs for monel metal at the high-frequency ends are above the corresponding graphs for steel. As the high-stress graphs for monel metal extend to the right, however, they turn downward and cross the corresponding graphs for steel. Whereas at high frequencies the rate of net damage is greater for steel than for monel metal, at low frequencies the reverse is true. A similar relationship would be observed if the Type 10 diagrams for aluminum bronze(13) and steel were superimposed. At low cycle frequency and high stress, therefore, in some cases at least, the net pitting rate is greater for the less generally corrodible of two metals.

## Intercrystalline Corrosion of Metals Under Stress

The relationship illustrated by the superimposed diagrams in Figs. 4 and 6 probably is not unusual. It seems probable that slight intensity of general corrosion accompanied by high stress range of low frequency generally favors a high rate of net damage. Such conditions tend to cause sharp, deep pitting. They also are favorable to the selective type of pitting known as intercrystalline corrosion.

Intercrystalline failure of metals under stress, as in season cracking of brass or the erroneously-called "caustic embrittlement" of boiler steel, probably is due to corrosion pitting differing only in location from ordinary sharp, deep corrosion pitting. The general conditions that favor intercrystalline corrosion are (1) high stress, (2) low cycle frequency, (3) slight general corrosion, and (4) very little accumulation of corrosion products. These conditions may cause sharp, deep corrosion pitting, whether or not the pitting is chiefly on the grain boundaries.

## "Determining Factors" in Stress Corrosion

U. R. Evans in his book on corrosion shows that the rate of corrosion, though affected by many factors, may depend chiefly on one "determining factor." The determining factor may be the rate at which oxygen diffuses to the cathodic areas and thus causes cathodic depolarization. Or the determining factor may be the rate of depolarization of the anodic areas. For ordinary steels in water, Evans says that the determining factor is usually the rate of cathodic depolarization. The differences in form of the diagrams discussed in the present paper probably are due in part to differences in the determining factor.

When the rate of cathodic depolarization is the determining factor in stress corrosion, the increasing actual stress in the (anodic) corrosion pits would have little effect in accelerating net damage. The rate of net damage, therefore, would depend on the initial stress in the cathodic areas. The straight-line logarithmic relationship between stress and rate of net damage obtained with ordinary steels in carbonate water may represent cathodic limitation throughout the entire field of the diagram. The narrower Type 11 (*a*) diagram obtained with monel metal and aluminum bronze may represent anodic limitation. The transition diagram obtained with steel in distilled water may represent a transition from anodic to cathodic limitation. At the high stress, high-frequency end, the diagram may represent anodic limitation. At the low-stress, low-frequency end, the diagram may represent cathodic limitation.

Two other factors in stress corrosion may vary so greatly in relative importance that either one or the other may be "determining." Stress may accelerate corrosion, not only by increasing the permeability of the film and corrosion products, but also by increasing the solution pressure of the metal. The relative importance of these two factors probably varies with the kind of metal, the intensity of general corrosion, the cycle frequency and the stress. Under severe general corrosion, the determining factor may be the influence of stress on the coating. Under slight general corrosion, at low cycle frequency, and especially under high stress, the determining factor may be the influence of stress on the solution pressure of the metal.

The increasing actual stress at the bottoms of corrosion pits tends to accelerate, and accumulation of corrosion products tends to retard, the progress of net damage. When general corrosion is slight, therefore, the retarding factor is slight, and the accelerating factor possibly has its full effect. It seems possible thus to account for the variations in form of the Type 10 and Type 11 (*a*) diagrams and especially to account for the fact that net damage at low cycle frequency may be greater under slight than under severe general corrosion.

## ACKNOWLEDGMENTS

Acknowledgment is due to the following personnel of the Naval Engineering Experiment Station: to Captain H. R. Greenlee, U. S. N., Officer in Charge, for interest and cooperation; to Mr. W. C. Stewart, Associate Metallurgist, for valuable cooperation and assistance; to Messrs. A. P. Vandermast and H. N. White, laboratorians, for assistance in the experiments.

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(3) D. J. McAdam, Jr., "Corrosion-Fatigue of Non-Ferrous Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 102 (1927).

(4) D. J. McAdam, Jr., "Fatigue and Corrosion-Fatigue of Metals," *Proceedings*, Internat. Congress Testing Mats., Vol. 1, p. 305 (1927).

(5) D. J. McAdam, Jr., "Fatigue and Corrosion-Fatigue of Spring Material," *Transactions*, Am. Soc. Mechanical Engrs. (1929), A. P. M. 511, 45.

(6) D. J. McAdam, Jr., "Corrosion of Metals as Affected by Time and by Cyclic Stress," *Proceedings*, Inst. Metals Division, Am. Inst. Mining and Metallurgical Engrs., p. 571 (1928).

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(10) D. J. McAdam, Jr., "Influence of Cyclic Stress on Corrosion," *Technical Publication No. 329*, Am. Inst. Mining and Metallurgical Engrs., Inst. Metals Division (1930).

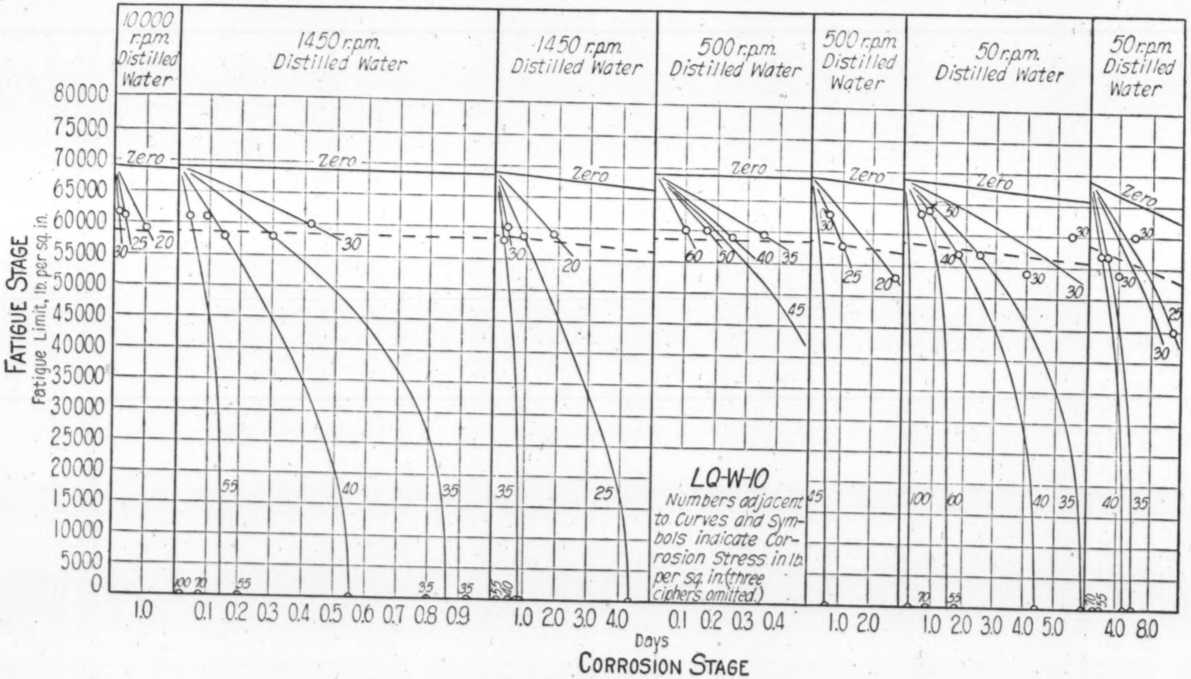
(11) D. J. McAdam, Jr., "The Influence of Stress Range and Cycle Frequency on Corrosion," *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part II, p. 411 (1930).

(12) D. J. McAdam, Jr., "Stress and Corrosion," Internat. Congress for Applied Mechanics, Stockholm, August, 1930.

(13) D. J. McAdam, Jr., "Influence of Stress on Corrosion," Am. Inst. Mining and Metallurgical Engrs., *Technical Publication No. 417*, April, 1931.

(14) D. J. McAdam, Jr., "Stress Corrosion of Metals," *Proceedings*, Internat. Assn. Testing Mats. (1931).

(15) B. P. Haigh, "Experiments on Fatigue of Brasses," *Journal*, Inst. Metals, Vol. 18, p. 55 (1917).



LQ-W-10
Numbers adjacent to Curves and Symbols indicate Corrosion Stress in lb. per sq. in. (three ciphers omitted.)

FIG. 1.—Type 5 Diagrams, Nickel Steel in Distilled Water.

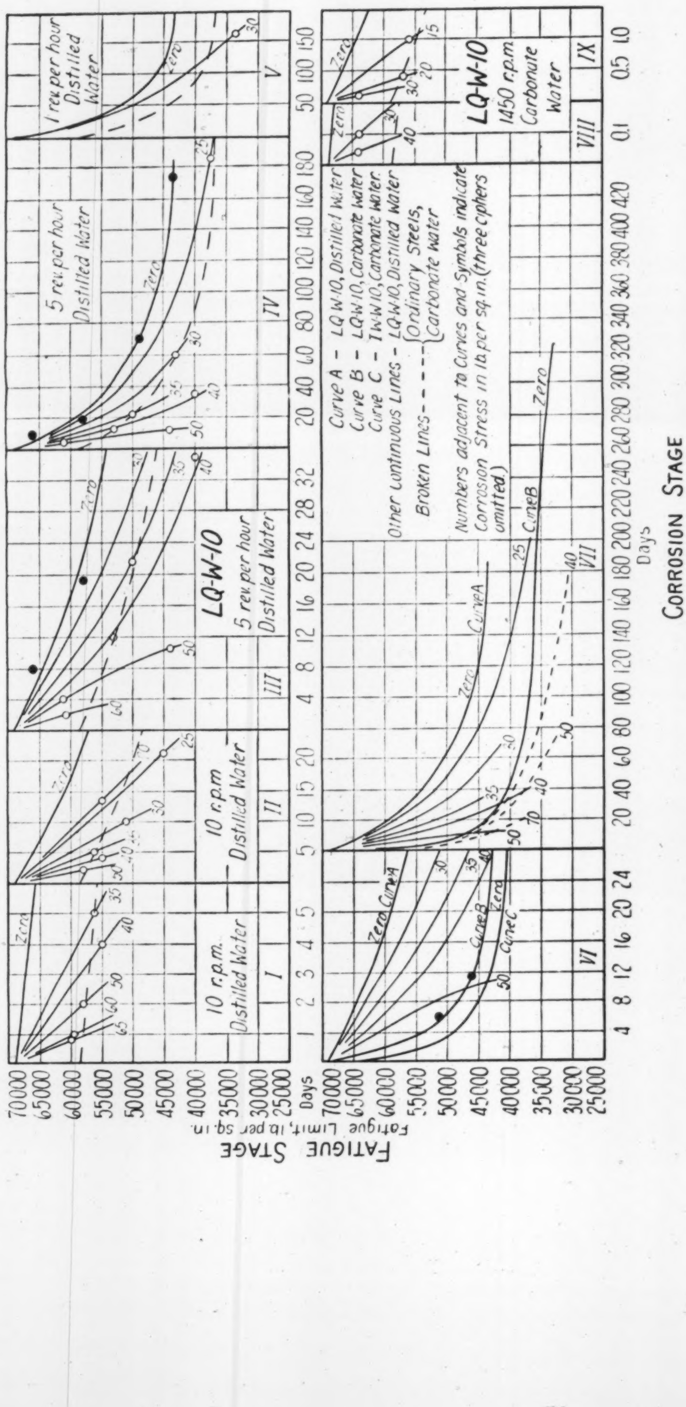


FIG. 2.—Type 5 Diagrams, Nickel Steel, Distilled Water and Carbonate Water.

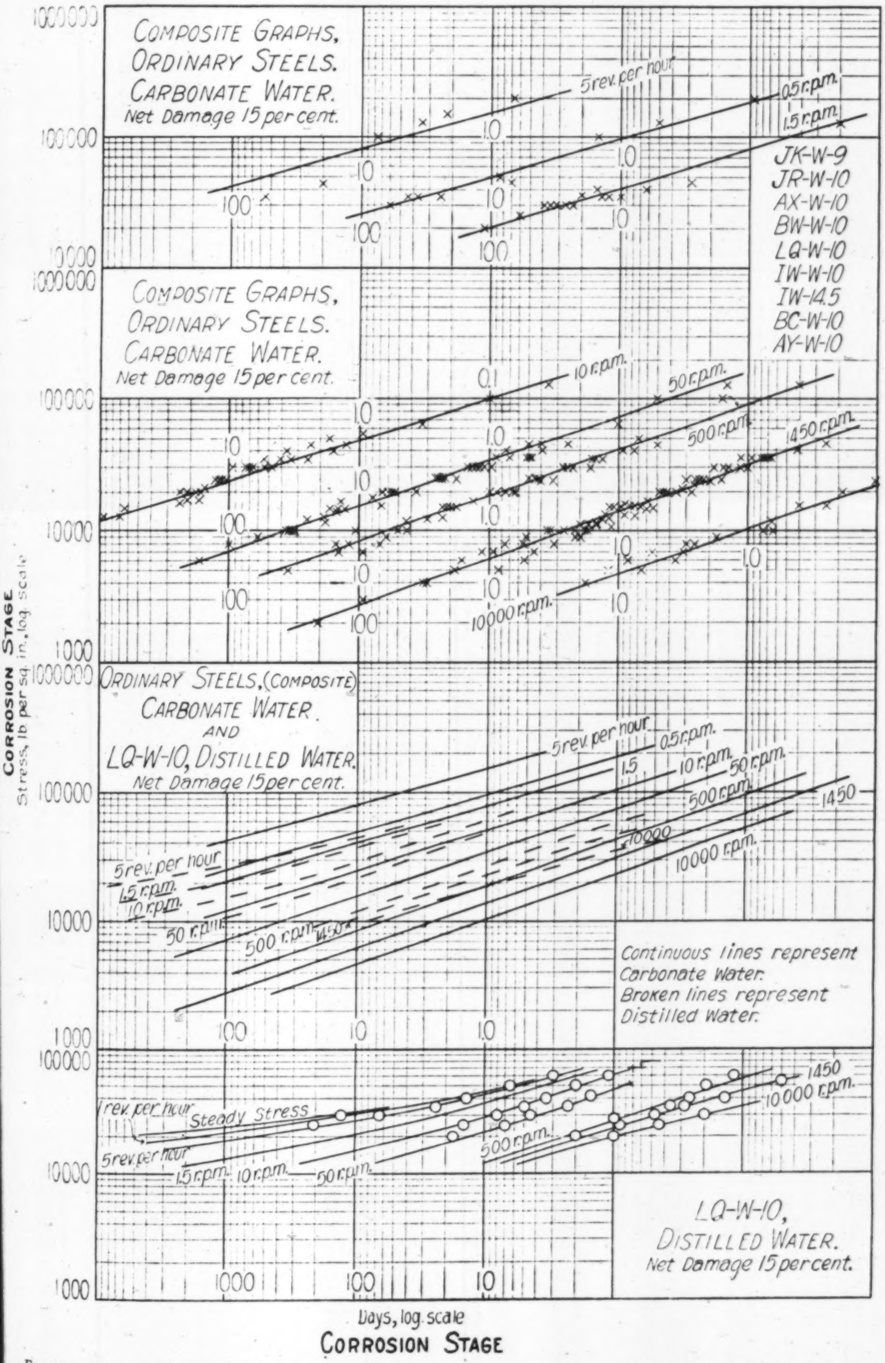


Fig. 3.—Type 11 (*a*) Diagrams, Steels in Carbonate Water and in Distilled Water.

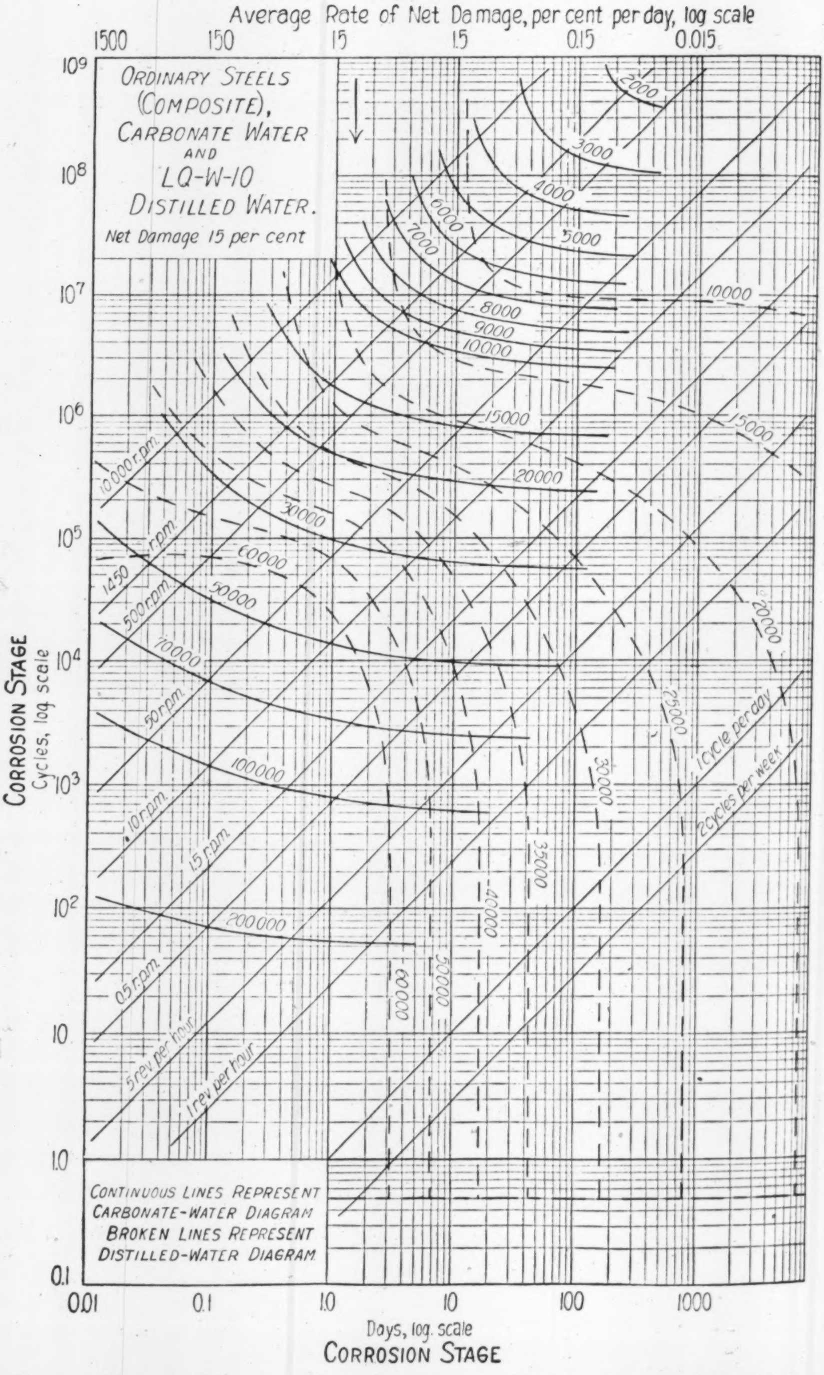


FIG. 4.—Type 10 Diagrams, Steels in Carbonate Water and in Distilled Water.

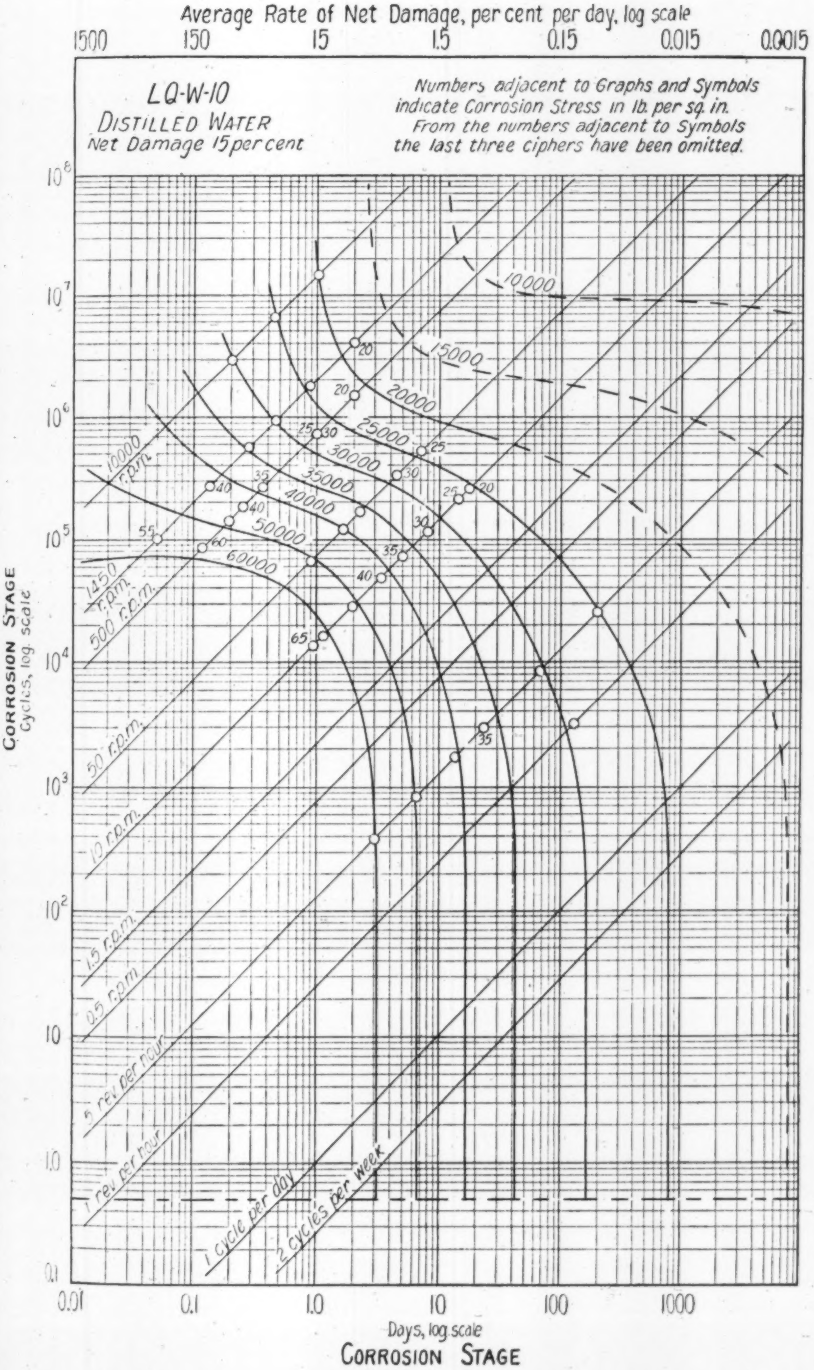


FIG. 5.—Type 10 Diagrams, Nickel Steel, Distilled Water.

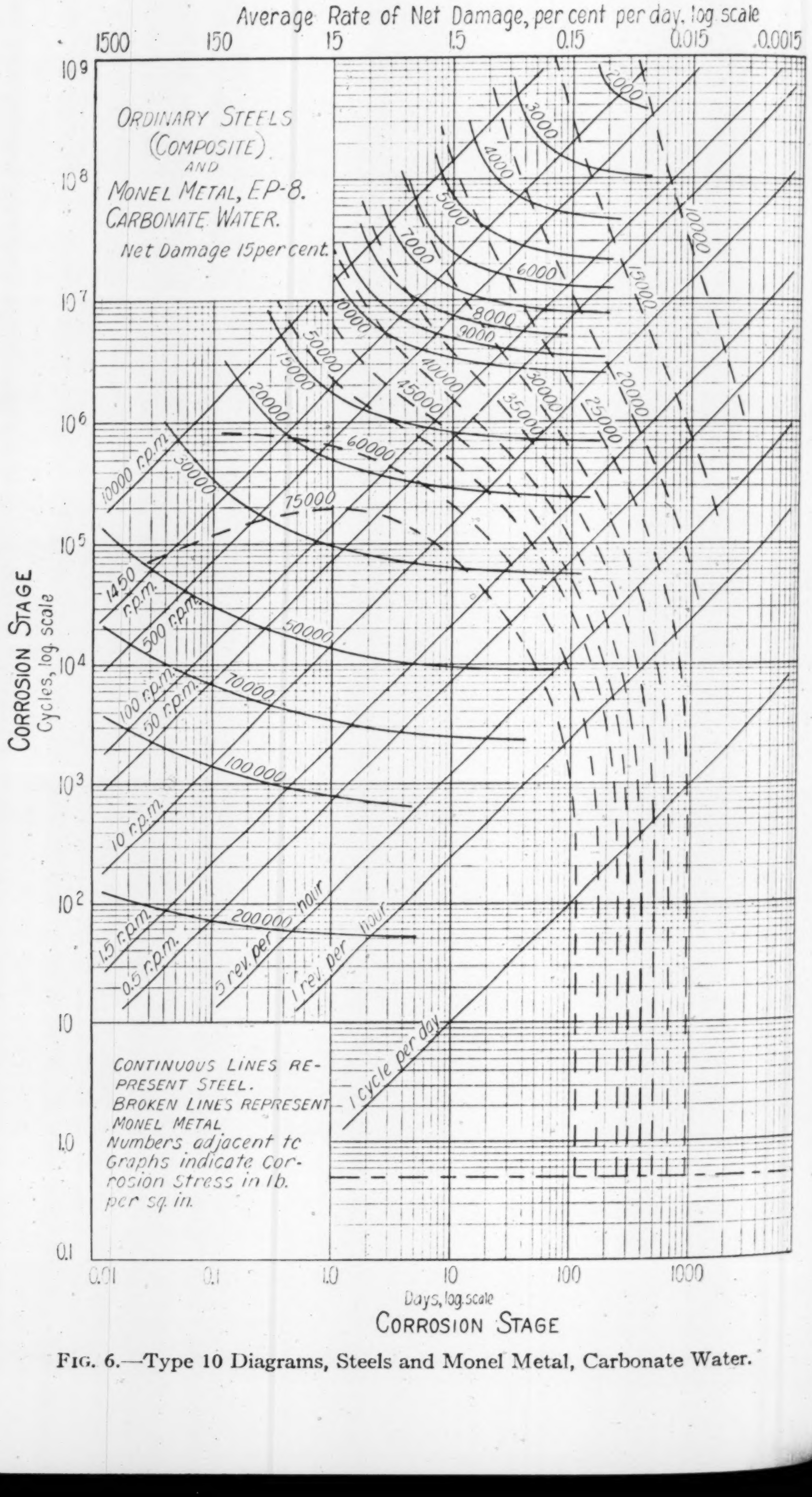


Fig. 6.—Type 10 Diagrams, Steels and Monel Metal, Carbonate Water.

# OXYGEN AS A FACTOR IN SUBMERGED CORROSION[1]

By Edward C. Groesbeck[2] and Leo J. Waldron[3]

## Synopsis

This paper is a progress report covering the first phase of an investigation being carried out at the U. S. Bureau of Standards on the rôle which dissolved oxygen plays in submerged corrosion of metals, under any selected set of controlled conditions. The range of dissolved oxygen concentrations covered in this investigation includes not only the amounts ordinarily found in waters and other liquids but also much higher amounts than have been used by previous investigators.

A laboratory apparatus of the continuous flow type was developed for this investigation, and was so constructed that those parts which come in contact with the corroding liquid are only of hard or soft rubber or Pyrex glass. The liquid was circulated through the apparatus and past the stationary specimens by means of a rotary pump.

In this phase of the work specimens of four low-carbon iron and steel sheet materials were immersed in city (tap) water for 1, 3 and 7 days and for 2 days in distilled water, having six different pH values and dissolved oxygen concentrations ranging from about 5 to 25 cu. cm. per liter. The temperature of the waters was maintained at 30° C. and the flow at about 15 feet per minute. The extent of the corrosion produced was determined by the loss-in-weight method.

For the city water, an increase in the concentration of dissolved oxygen produced, in general, an increase in the corrosion rate, and the weight losses approached a constant value with time. Two distinct types of curves were obtained from the results with the distilled water at the different pH values. The corrosion rates of the test materials, as represented by these curves, at the different dissolved oxygen and hydrogen-ion concentrations have been discussed on the basis of the electrolytic corrosion theory.

## Introduction

A striking feature in the results of the cooperative laboratory corrosion tests made a few years ago, under the sponsorship of the Society's Committee B-3 on Corrosion of Non-Ferrous Metals and Alloys, on a series of metals was the lack of concordance in the test results reported by the various cooperating laboratories. It was stated in the report of this committee[4] that the degree of aeration is of vital importance and that this factor had not been controlled sufficiently closely in the tests. It is obvious that the importance of the rôle which dissolved oxygen plays had not been fully realized at the time the tests were started, especially as the tests were to be carried out under as nearly comparable conditions as possible at the different cooperating laboratories.

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[1] Publication approved by the Director of the Bureau of Standards of the U. S. Department of Commerce, Washington, D. C.

[2] Metallurgist, U. S. Bureau of Standards, Washington, D. C.

[3] Assistant Scientist, U. S. Bureau of Standards, Washington, D. C.

[4] Report of Committee B-3 on Corrosion of Non-Ferrous Metals and Alloys, *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 281 (1927).

It is a well-established fact that the attack which a corrosive liquid produces upon an immersed metal may be markedly dependent upon the degree to which that liquid is aerated, or in other words, the concentration in the liquid of dissolved and gaseous oxygen. Rawdon and one of the authors[1] showed by numerical data that aeration of the liquid, in which a series of commercial copper-nickel specimens were tested by the simple immersion and repeated immersion methods, produced a pronounced accelerating effect on the corrosion of these specimens.

One factor which may determine the rate at which iron and steel corrode when submerged in a liquid medium is the supply of oxygen available at the metal surface. This supply may be effected by either (1) diffusion or convection, or both, from the liquid surface exposed to the overlying atmosphere or gas, or (2) the direct introduction into the liquid, by aeration or otherwise, of oxygen gas or a gaseous mixture containing oxygen followed by a transfer to the metal surface by means of diffusion or convection. This rate of corrosion is known to be influenced by certain other factors such as film formation, concentration of hydrogen ions or other ions in the liquid medium, and the velocity of movement of the liquid medium past the metal. It is essential, therefore, to understand the mechanism by which the oxygen present, either in the dissolved or free state, in the liquid medium promotes the corrosion of iron and steel, with or without the influence of one or more other factors.

A number of investigators have studied the effect of dissolved oxygen upon the corrosion of metals by waters and acid solutions, but they moved the metal specimens and kept the corroding medium practically stationary. A few years ago, Speller suggested a continuous flow apparatus, in which the metal specimens would be held suspended in a stream of flowing liquid.[2] Fraser, Ackerman and Sands[3] were the first to develop an apparatus of this type in connection with their quantitative study of the submerged corrosion of monel metal in sulfuric acid.

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[1] H. S. Rawdon and E. C. Groesbeck, "Effect of the Testing Method on the Determination of Corrosion Resistance," Bureau of Standards *Technologic Paper No. 367*, p. 442 (1928).

[2] F. N. Speller, "Corrosion: Causes and Prevention," McGraw-Hill Book Co., Inc., New York City, p. 228 (1926).

[3] O. B. J. Fraser, D. E. Ackerman and J. W. Sands, "Controllable Variables in the Quantitative Study of the Submerged Corrosion of Metals," *Industrial and Engineering Chemistry*, Vol. 19, p. 332 (1927).

An apparatus based on Speller's main idea and constructed of such materials that the only surfaces with which the corroding medium may come in contact are of hard or soft rubber or Pyrex glass was developed at the U. S. Bureau of Standards for studying the submerged corrosion of metals and the effects of the concentrations of dissolved oxygen, hydrogen ions, and other ions and salts, and of the velocity of flow of liquid past the metal under test.

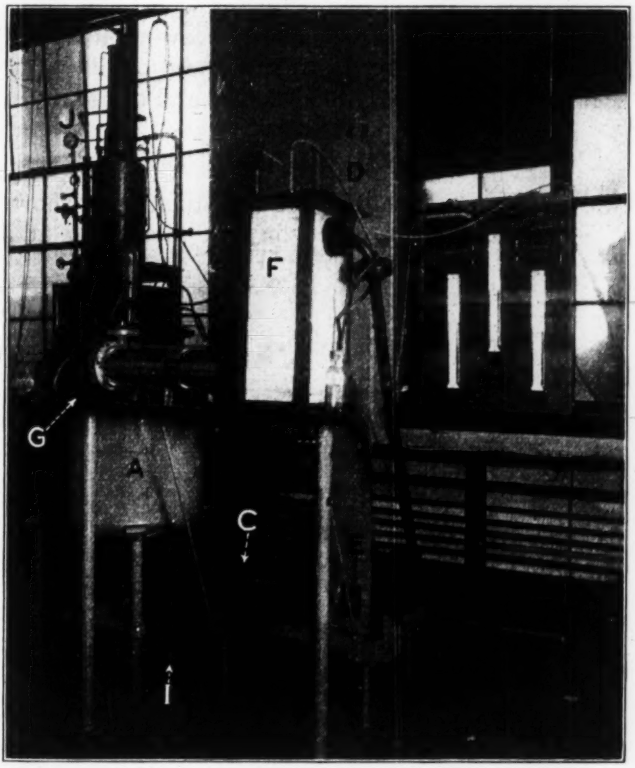

FIG. 1.—Continuous Flow Apparatus.

*A*, Rubber-lined steel storage tank (50 gal.).
*B*, Specimen chamber.
*C*, Hard rubber rotary pump.
*D*, Differential manometer.
*E*, Orifice meter.
*F*, Auxiliary rubber-lined steel storage tank.
*G*, Sampling tap.
*H*, Flowmeters.
*I*, Drain
*J*, Water still.

In published accounts of investigations in which the effect of oxygen in submerged corrosion was considered, the greatest amount of dissolved oxygen used was not over about 9 cu. cm. per liter. The solubility of oxygen in water at 20° C. and ordinary atmospheric pressure is approximately 30 cu. cm. per liter. Ordinary waters do not contain, at 20° C., more than approximately 5 cu. cm. per liter of dissolved oxygen absorbed from the air.

This paper is a progress report on the first phase of this study, which deals especially with the behavior of low-carbon steel and iron when immersed in flowing city (tap) water and distilled waters, each having a definite temperature, velocity, pH and oxygen content.

## APPARATUS

The apparatus developed for this investigation is shown in Figs. 1 and 2. The operation is based on the circulation at a definite velocity of the liquid from a storage tank, *A*, through $\frac{3}{4}$-in. bore hard rubber tubing to a specimen chamber, *B*, which contains the materials being tested, and thence back to the tank by means of a rotary pump, *C*. The liquid in the tank was maintained at 30° C. ± 0.5° C. by means of heaters and a thermoregulator. A stream of air, or a gaseous mixture containing sufficient oxygen to impart to the liquid the desired concentration of dissolved oxygen, was bubbled through the liquid by means of a glass tube extending through the tank cover. A calibrated differential pressure manometer, *D*, and an orifice meter, *E*, served to indicate the rate of flow of the liquid through the chamber. A velocity of about 15 ft. per minute was employed.

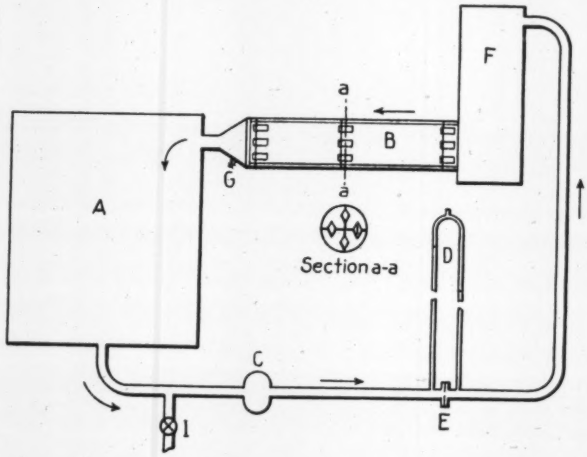

Fig. 2.—Diagrammatic Sketch Showing the Principal Parts of the Apparatus.

The chamber $B$ consisted of a Pyrex glass tube, 4 in. in inside diameter and 24 in. long, mounted in a cylindrical frame made of a cast aluminum alloy and provided with eight rectangular slots to permit inspection of the contents. A rack, $3\frac{3}{4}$ in. in diameter and 20 in. long, constructed of 0.08-in. glass rod, served to hold the specimens. The twelve specimens were placed in a vertical position and with the long side of each horizontal, in V-shaped receptacles placed 90 deg. apart in groups of four, at both ends and the middle of the rack. The connection between the chamber and storage tank was detachable to permit the introduction or removal of the rack, with the specimens in place. The frame was held in place by means of bolts, and soft rubber gaskets were used at both ends of the tube to form water-tight joints.

At the inlet end of the chamber was an auxiliary tank, $F$, which served to remove from the stream of flowing liquid any bubbles of air or other gas, and to reduce eddy currents. The need for this tank became apparent after a cone-shaped adapter similar to the one shown in part in Fig. 1 at the exit end of the specimen chamber had been employed during the early part of this study to connect the tubing (at the right in Fig. 1) to the chamber. The use of this adapter permitted the formation of undesirable eddy currents, and irregular results were consequently obtained for the specimens at the inlet end. The air or gas mixture was passed directly into the liquid in $F$ by means of a glass aerating device extending through the top of this tank and into the liquid, in order to maintain the desired dissolved oxygen content. A glass tube in the top of $F$ served as a gas outlet.

TABLE I.—COMPOSITION OF SAMPLES.

| Designation | Kind of Material (in sheet form) | Chemical Composition, per cent | | | | | | | | | Thickness, in. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Carbon | Manganese | Sulfur | Phosphorus | Silicon | Copper | Molybdenum | Nickel | Chromium | |
| A | Low-carbon open-hearth iron[a] | 0.01 | 0.03 | 0.020 | 0.003 | <0.01 | 0.03 | .... | 0.02 | 0.01 | 0.030 |
| B | Low-carbon cold-rolled steel[a] | 0.10 | 0.37 | 0.025 | 0.013 | <0.01 | 0.08 | .... | 0.05 | 0.02 | 0.060 |
| C | Low-carbon steel, containing copper[a] | 0.12 | 0.48 | 0.052 | 0.092 | 0.02 | 0.22 | .... | 0.03 | 0.01 | 0.030 |
| D | Low-carbon open-hearth iron, containing copper and molybdenum[b] | 0.04 | 0.13 | 0.035 | 0.006 | <0.01 | 0.43 | 0.07 | .... | .... | 0.125 |

[a] Analyses made by Bureau of Standards.
[b] Manufacturer's analysis.

On the under side of the cone-shaped adapter at the exit end of the specimen chamber is a glass stop-cock, $G$, through which samples of the water were drawn off for the determination of dissolved oxygen and pH.

## TEST MATERIALS

The metals tested in this investigation have the composition and thickness shown in Table I.

The specimens of these metals were 1 by 2 in., with the length cut in the direction of rolling. The surface of the specimens was ground free from scale and surface defects on a belt grinder and was finished by hand with Manning 1G emery paper.

All of the metals were used with distilled water, and metals A, B and C also with city water.

The city water used for the first series of tests contained initially about 5 cu. cm. dissolved oxygen per liter at room temperature (about 20° C.) and had a pH value of 7.8 to 8.0. The distilled water was supplied by a still installed especially for this work, and had under ordinary conditions of exposure and at room temperature, a dissolved oxygen content of about 1 cu. cm. per liter and a pH value of 6.8 to 7.0. The maximum dissolved oxygen content, obtained by saturating the distilled water with oxygen gas, was determined to be about 25 cu. cm. per liter at 30° C. (the temperature used in the corrosion tests).

## METHOD

The procedure followed in making the tests was first to bring the temperature of the water in the nearly filled tank *A* to 30° C., at which it was automatically maintained. At the same time, air, or other gas, was bubbled into the water to bring the dissolved oxygen content to the desired point. The specimens, which had previously been washed with alcohol, benzol and ether and weighed to the nearest 0.1 mg., were placed in the apparatus. Three specimens of each metal, or 12 in all, were used. Enough water, into which the gas also had been bubbled, was next added to fill the specimen chamber. The pump was started and kept in operation throughout each test.

Gas mixtures composed of air and oxygen, air and nitrogen, or oxygen and nitrogen were prepared with the aid of two flowmeters (*H*, Fig. 1) in the proper proportions for giving to the water the desired concentration of dissolved oxygen. The third flowmeter was used to control the proportion of carbon dioxide required to produce in the distilled water pH values of 4.5 and 5.0. To obtain pH values of 5.5 and 6.0, air or the gas mixture was first passed through a bottle containing a saturated solution of sodium bicarbonate, or a 50–50 mixture of saturated solutions of sodium carbonate and bicarbonate, whereby sufficient, yet very small, amounts of $CO_2$ were introduced before the air or gas mixture was admitted to the water.

To obtain water with a pH value of 8, a sufficient amount of a very weak solution of sodium hydroxide (about 0.01 *N*) was added to the water.

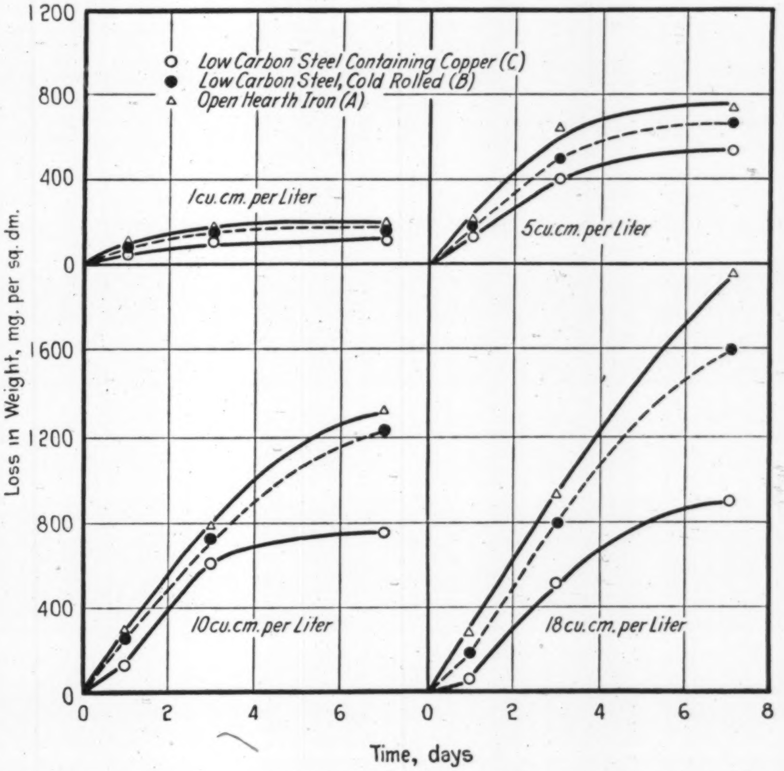

FIG. 3.—Corrosion Loss in City Water with Different Oxygen Concentrations, at 30° C.

The individual points in these diagrams were calculated from the curves for 1-, 3-, and 7-day runs.

The determination of the dissolved oxygen content was made by the Winkler method[1]. The Levy method, used by many foreign investigators for the past 50 years[2, 3, 4 5] was also employed in the earlier work as a check on the Winkler method. The results were in good agreement, though those obtained by the Levy method usually indicated approximately 0.2 to 0.3 cu. cm. per liter more dissolved oxygen than by the Winkler method. The dissolved oxygen content was calculated to standard conditions of temperature and pressure. The pH values were determined by the isohydric colorimetric indicator method.[1] Samples of the water were taken at the beginning and end of each run and at intervals between.

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[1] Appendix to Report of Society Representatives on the Joint Research Committee on Boiler Feed Water Studies, *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part I, p. 178 (1929).

[2] A. M. Buswell and W. V. Gallaher, "Determination of Dissolved Oxygen in the Presence of Iron Salts," *Industrial and Engineering Chemistry*, Vol. 15, p. 1186 (1923).

[3] E. A. Letts and R. F. Blake, "On a Simple and Accurate Method for Estimating Dissolved Oxygen in Fresh Water, Sea Water, Sewage Effluents, etc.," *Proceedings*, Royal Dublin Soc., Vol. 9, p. 454 (1899–1902).

[4] P. Kay, "Simple Methods for the Determination of Dissolved Oxygen in Waters, Effluents, etc.," *Chemical News*, Vol. 110, p. 49 (1914).

[5] Levy, *Annales*, Observatoire de Mort-souris, p. 3101 (1894).

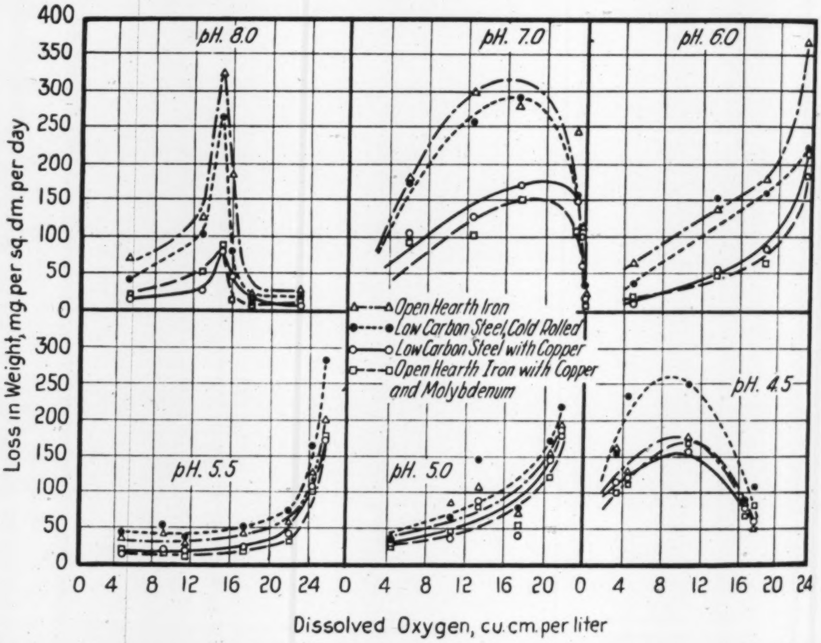

FIG. 4.—Relation of Rate of Corrosion and Concentration of Dissolved Oxygen in Distilled Water for Different pH Values.

Individual points represent observations in 48-hour runs.

At the end of a test run, the metal specimens were removed and freed from adhering corrosion products. During the early part of the work, a chemical method was used, and later an electrolytic method was substituted. In the former method, the specimens were immersed in a warm 10-per-cent solution of ammonium citrate for 10 to 40 seconds. The time of immersion depended on the degree of corrosion and was the same for all specimens in any given test. In the latter method, the specimen was made the cathode in normal sulfuric acid, with a platinum wire coil as the anode.[1] Later, the platinum anode was replaced by a section of a Duriron pipe, about 3 in. in diameter. It was determined by experiment that with a Duriron anode, a clean iron specimen lost only about 0.0003 g. in a one-hour treatment. The electrolytic cleaning was limited to 15 minutes or less, as the corrosion products were completely removed in that time. The specimens were cleaned, washed and dried in the same manner as before the test, and reweighed, to determine the loss in weight due to corrosion.

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[1] E. H. Fawcett and S. F. Acree, "Isohydric Indicator Methods for the Accurate Determination of pH in very Dilute Solutions," *Journal of Bacteriology*, Vol. 17, p. 171 (1929).

## RESULTS

The results are shown graphically in Figs. 3 and 4. Data from a number of duplicate runs with distilled water confirmed the curves of Fig. 4.

The degree of reproducibility obtained in the test results is an important consideration in any testing method. The average variation of the individual losses in weight from the mean for specimens of the same material in a given test run was not over 14 per cent and mostly under 7 per cent when the loss in weight amounted to 200 or more mg. per sq. dm., and not over 50 per cent when the loss in weight was only a few milligrams.

## DISCUSSION OF RESULTS

The following interpretations of the results are based upon the well-known principles of electrolytic corrosion.[2, 3, 4, 5, 6] It is of interest to note that some of the results reported here are in good agreement with those obtained by previous investigators.

### *City water:*

It is clearly indicated in Fig. 3 that the rate of corrosion for the three materials tested in city water, with a pH value of 7.8 to 8.0, increased with the concentration of dissolved oxygen, within the range from 1 to 18 cu. cm. per liter, and with the duration of the test up to 7 days. The increase in corrosion rate for the copper-bearing low-carbon steel, material C, was, however, appreciably less than that for the other two materials when the oxygen concentration exceeded 5 cu. cm. per liter. The rate of corrosion for material C in the 1-day runs with 10 and 18 cu. cm. of oxygen per liter was also slower than that for the lower oxygen concentrations, which fact is in agreement with the decidedly less acute slope shown in Fig. 4 for material C in distilled water with pH values of 8.0 and 7.0, as compared with the slopes for materials A and B at the same oxygen concentrations. The rate of increase in weight loss per day for material C in the 7-day runs with increasing oxygen concentration remained quite the same for the 1 and 5 cu. cm. of oxygen per liter but steadily decreased as the oxygen content rose to 18 cu. cm. per liter.

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[1] B. McCollum and K. H. Logan, "Electrolytic Corrosion of Iron in Soils," Bureau of Standards *Technologic Paper No. 27*, p. 7 (1913).

[2] W. G. Whitman, "Corrosion of Iron," *Chemical Reviews*, Vol. 2, p. 419 (1925).

[3] W. G. Whitman, R. P. Russell and V. J. Altieri, "Effect of Hydrogen-ion Concentration on the Submerged Corrosion of Steel," *Industrial and Engineering Chemistry*, Vol. 16, p. 665 (1924).

[4] R. E. Wilson, "The Mechanism of the Corrosion of Iron and Steel in Natural Waters and the Calculation of Specific Rates of Corrosion," *Industrial and Engineering Chemistry*, Vol. 15, p. 127 (1923).

[5] W. G. Whitman and R. P. Russell, "The Submerged Corrosion of Iron," *Journal*, Soc. Chemical Industry, Vol. 43, p. 193T (1924).

[6] J. W. Shipley and I. R. McHaffie, "The Relation of Hydrogen-ion Concentration to the Corrosion of Iron," *Canadian Chemistry and Metallurgy*, Vol. 8, p. 121 (1924).

It has been shown[1, 2] that variations in the hydrogen-ion concentration of the corroding liquid, within the range from pH 9.5 to 4.5, do not change the corrosion rate, since the liquid film adjacent to the metal maintains a constant pH value because of the solubility of ferrous hydroxide, and hence acts as a protective film. The rate of corrosion in natural waters is determined by the speed at which oxygen diffuses to the cathodic area and by the degree of protection which the resultant film offers. Therefore, the extent of corrosion of iron in a natural water depends upon the concentration of dissolved oxygen. A protective rust film is built up on the iron with continued exposure. The curves for the low oxygen concentrations in Fig. 3 appear to be in good agreement with these statements. The fact that the copper content in material C did not produce any clearly defined improvement in corrosion resistance for the oxygen concentrations of 1 and 5 cu. cm. per liter seems to be in accord with the observations made a few years ago by the Society's Committee A-5 on Corrosion of Iron and Steel[3] that in the case of steel totally immersed in city, brackish and acid mine waters the presence of copper is not a factor in the life of the materials. The city water and the brackish water contained, as determined by analysis, no more than about 3.5 cu. cm. of oxygen per liter at 28° C.

*Distilled water:*

The results obtained with distilled water at different hydrogen-ion and oxygen concentrations are given in Fig. 4. The plotted

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[1] W. G. Whitman, R. P. Russell and V. J. Altieri, "Effect of Hydrogen-ion Concentration on the Submerged Corrosion of Steel," *Industrial and Engineering Chemistry*, Vol. 16, p. 665 (1924).

[2] W. G. Whitman and R. P. Russell, "The Submerged Corrosion of Iron," *Journal*, Soc. Chemical Industry, Vol. 43, p. 193T (1924).

[3] Report of Subcommittee V on the Total Immersion Tests of Committee A-5 on Corrosion of Iron and Steel, *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part I, pp. 226 and 232 (1924); also, Vol. 28, Part I, p. 160 (1928).

results show two distinct types of curves. One follows a sloping course with the crest or maximum at a point approximately midway in oxygen concentration (curves for pH 8 and 7). The observations made were not sufficiently numerous to determine the exact conditions for maximum corrosion, and the maxima shown in these curves should be regarded as being only indicative of the trend of the curves. The other type follows an approximately horizontal course starting at the low oxygen content and rising more or less sharply when approaching the high oxygen content (curves for pH 5.5 and 5). The curves for pH 6.0 appear to lie between these two types.

It should be borne in mind that these curves do not represent a progressive corrosion, or increase in corrosion loss, of the metal with time, as was the case in Fig. 3, but the average rate of the corrosion produced in 48 hours at any given oxygen content and pH value. It is probable that an invisible oxide film was present on the surface of the specimens when they were put in the test and that the corrosion started at places where resistance to corrosive attack was weak because of discontinuities in this film, or because of particles of foreign matter from the liquid adhering to the film, or some other factor of a similar nature.

*pH value 8.0.*—In this slightly alkaline water the left branch of the curves is due to anodic corrosion of the iron, with the ultimate formation of insoluble ferric hydroxide, and to cathodic depolarization at the low oxygen concentrations. As the quantity of ferric hydroxide increases, the oxygen concentration cell effect enters and becomes more pronounced with an increase in the oxygen content, hence the sharp rise in the corrosion rate. The specimens tested in water having an oxygen content of 15 cu. cm. per liter were found to be pitted to a considerable extent. When the oxygen concentration was higher, it was great enough to exert an oxidizing effect on the film and to repair breaks in it and little or no corrosion was produced. Hence, the curves show a sharp drop from the crest to a horizontal direction toward the high oxygen end.

*pH value 7.0.*—Since this water is "neutral," the anodic corrosion and cathodic depolarization and the corrosion due to the oxygen concentration cells were greater, as shown by the sharper rise in the curves for the low oxygen content. The sharp drop in the curves was shifted further to the high oxygen end. The specimens tested in water having an oxygen content of 15 cu. cm. per liter were more deeply pitted than those in water having an oxygen content of either 6 or 24 cu. cm. per liter.

*pH value 6.0.*—The same general tendencies hold as for pH value 7, except that the zone of maximum corrosion is shifted still further to the right. The specimens tested with 24 cu. cm. of oxygen per liter showed appreciably more adherent rust than those at lower oxygen concentrations.

*pH value 5.5.*—The curves for this pH value differ from those for pH 6.0 and 5.0 by having relatively low corrosion rates at oxygen concentrations below 20 cu. cm. per liter and a marked increase in corrosion at higher oxygen concentrations. (Probably at still higher oxygen concentrations, such as might be obtained at increased pressure, the curves would reach maxima and then recede.) According to the electrolytic theory of corrosion, the anodic process, wherein the metal actually goes into solution, must always be accompanied by an equivalent cathodic process. The latter may consist of either direct evolution of hydrogen or reduction of dissolved oxygen. An increase in hydrogen-ion concentration (decrease in pH) will normally favor the former, while an increase in oxygen content will favor the latter. The general increase in the rate of corrosion, at any given pH value, with increased oxygen content, up to a maximum, is an evidence of the latter cathodic process. The drop in the corrosion rate beyond the maximum is probably due to a passifying effect produced by the increased oxygen content upon the anode, whereby the anodic process is retarded. To test this explanation experimentally, it would be necessary to study separately the anodic and cathodic processes accompanying corrosion under given conditions.

It is difficult to explain the reduction in corrosion rate for the lower oxygen concentrations at a pH value of 5.5. It is, at least, possible that the incipient evolution of hydrogen at this pH value might interfere with the cathodic reduction of oxygen. In that case, the results obtained for this pH value would represent a transition from one cathodic process to another and indicate that up to 20 cu. cm. of oxygen per liter neither process readily occurs. With an increase either in oxygen content or acidity (for example, to pH value 5.0), the corresponding cathodic process is accelerated and corrosion is increased.

It is also possible that at a pH value of 5.5 there is an incipient tendency for the film of ferrous hydroxide formed at the anodic portions to be dissolved, and hence the formation of oxygen concentration cells, which accelerate corrosion and expecially pitting, to be retarded. This explanation is confirmed by the fact that there was less pitting of the metal exposed at a pH value of 5.5 or lower than at a higher pH value. At best, it is difficult to offer a full explanation of these results without undertaking further experimental work in which the above-mentioned possibilities would be investigated.

*pH value 5.0.*—As this water had a higher hydrogen-ion concentration, the tendency for evolution of hydrogen gas was increased. This water was probably sufficiently acid to exert a solvent action on the ferrous hydroxide and thus to retard the formation of insoluble ferric hydroxide. Cathodic depolarization appears to have been the controlling factor as the oxygen concentration was increased. The specimens tested at 22 cu. cm. per liter were appreciably rusted, but not much pitted.

*pH value 4.5.*—Since the gaseous mixture used for aerating the water consisted of 25 per cent $CO_2$ and 75 per cent $O_2$, by volume, this water may be considered as a weak acid. The action of carbonic acid on iron is known to cause an evolution of hydrogen gas at about pH 5.4, although at this pH value it is only 6 per cent dissociated. However, it possesses sufficient acidity to neutralize the alkaline liquid film next to the metal. An increase in oxygen concentration increases the corrosion rate. The initial stages of the curves show a steady increase in the corrosion rate up to about 9 cu. cm. of oxygen per liter. Beyond this point, they show a steady decrease to a point, at about 18 cu. cm. of oxygen per liter, where the weight loss is even smaller than that at 2 cu. cm. of oxygen per liter. It seems reasonable to expect that the formation of carbonate of iron near the metal surface would tend to decrease the corrosion rate. The specimens at all oxygen concentrations showed complete absence of adherent rust layers.

*Acknowledgment.*—The authors desire to express their appreciation for the efficient aid rendered by the late Harold F. Grasse, Junior Scientist, Bureau of Standards, whose death occurred when the preparation of this paper was nearly completed.

# DISCUSSION

Mr. O. B. J. Fraser[1] (*presented in written form*).—The authors have undertaken a very important and very complex problem and it is to be hoped that their further studies will continue the clarification of the general problem of corrosion which those now reported have begun.

The initial results show not only the great necessity for exerting close control over oxygen concentration in all corrosion studies, but also that even with control of oxygen and other participants, as well as the other conditions of test, the progress of corrosion reactions is subject frequently to variations beyond control of the experimenter. This is brought out in the authors' discussion, on page 287, of the degree of reproducibility of results. As the weight losses decreased, the scatter among check determinations increased greatly. The paper by Fraser, Ackerman and Sands cited, shows a similar increase in scatter occurring in the case of the corrosion of monel metal by aerated sulfuric acid in an acid concentration range where one corrosion reaction is displaced by another of quite different nature.

The disconcerting manner in which weight losses vary when more than one of the active participants are varied is shown in Fig. 4 of the paper. A somewhat parallel situation has been found in the behavior of nickel in milk.[2] With limited input of air the rate of solution is lower in high than in low-acid milk, but with increased air supply the opposite is true. In the present case the variation of corrosion rates of iron with pH at constant oxygen input is also very erratic.

In seeking an explanation for the great differences in the shapes of the curves of Fig. 4 showing the relationship of corrosion rates to oxygen concentrations in short-time tests it would be well to consider the possible effects of hydrogen-ion concentration within the wide range studied upon the physical nature of the films of hydrated iron oxides formed on the test pieces. It is possible that the constant weight losses shown at pH 5.5 for oxygen concentrations varying from 4 to over 20 cu. cm. per liter may be because at or near this pH value the colloidal iron oxides may exist in a state of continuity most favorable to the formation of a film of maximum inhibitory capacity. At lower pH values, repair of the film becomes increasingly difficult because of increasing tendency to dissolve and at higher pH values it may be coagulated to some extent and thus be more pervious to oxygen.

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[1] Research Laboratory, International Nickel Co., Bayonne, N. J.

[2] R. J. McKay, O. B. J. Fraser and H. E. Searle, "Practical Application of Corrosion Tests: Resistance of Nickel and Monel Metal to Corrosion by Milk," *Technical Publication No. 192*, Am. Inst. Mining and Metallurgical Engrs. (1929).

The authors have drawn smooth curves in Fig. 4 for results of tests at pH value of 5.0 although the points for all four grades of material show the possible occurrence of a maximum in the vicinity of 14 cu. cm. of oxygen per liter followed by a minimum. Perhaps such a situation does exist, which would add something further to the complexity of the problem.

MR. F. N. SPELLER[1] (*presented in written form*).—In general, the data reported on influence of dissolved oxygen of the concentrations usually found at normal temperatures in tap water are in accord with previous work done here and abroad. At a constant rate of motion the diffusion of oxygen to the metal varies mainly with the physical character of the rust layer. A more highly oxidized and more resistant rust film probably forms at higher oxygen concentrations on copper-bearing steel and accounts in part for the slower rate of corrosion observed.

The results from distilled water tests (Fig. 4) at various pH values for the same oxygen concentration do not seem to be consistent with previous experience. If these data in whole or in part are confirmed by subsequent tests, the results may be of considerable significance. The distinctly lower rate of corrosion of copper-bearing steel formed in distilled water in a 48-hour test is in line with results reported from short-time tests by Chappell.[2] If there was an opportunity for the formation of an oxide film in air before the specimens were immersed, as the authors indicate, this would be more protective in the case of copper-bearing steel and this metal would, therefore, probably show greater initial resistance to corrosion in water than plain steel. Other tests in water over longer periods show little difference between iron and steel with and without copper.

We are glad to note that this interesting work will be continued. The results promise to throw more light on the mechanism of corrosion, particularly on the protective influence of various kinds of corrosion products. It is to be hoped that the composition of these products formed under various oxygen concentrations will be determined, if possible.

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[1] Director, Department of Metallurgy and Research, National Tube Co., Pittsburgh, Pa.

[2] E. L. Chappell, "Influence of Rust Film Thickness on Rate of Corrosion of Steels," *Industrial and Engineering Chemistry*, Vol. 19, p. 464 (1927).

# ACCELERATED CORROSION TEST OF SPRAYED MOLTEN METAL COATINGS APPLIED ON STEEL IN A $SO_2$-$CO_2$-AIR ATMOSPHERE

By Leopold Pessel[1]

## Synopsis

The relative resistance of various sprayed molten-metal coatings, applied on steel, against the attack of the corrosive agents occuring in flue gases was determined in an apparatus, in which this attack is considerably accelerated and in which uniform corrosion results are obtainable. Sprayed molten-lead coatings were found to be practically unattacked. Twenty-two other metals and metal combinations, applied in sprayed molten form, indicated lack of suitability for service under conditions similar to those developed in this apparatus.

Coatings produced by spraying molten metal have frequently been called upon, within recent years, to protect steel parts exposed to the attack by corrosive fumes containing mixtures of sulfurous and sulfuric acid. From the very beginning lead was accepted as the best metal available in sprayed molten form to meet this type of service. Sometimes, however, the supremacy of lead over some other sprayed metal is questioned. The present test was undertaken to establish a comparison between sprayed molten lead and a number of other sprayed metals by means of an accelerated method. It is not the intention to claim that the conditions obtained in this test, that is, the relative quantitative composition of the corrosive agents employed, are identical with those obtained under any specific service condition. This point must remain open for discussion. The results obtained, however, are so definite that it is held improbable that they may be reversed by changes of the relative composition of the various corrosive agents employed, such as may occur under different service conditions.

Figure 1 shows the longitudinal section of the apparatus used in these tests. A slightly inclined stone tank is provided with built-in stone blocks, suitably arranged to support glass plates serving as baffles and shelves. The specimens were suspended in the elongated sections of double glass loops. The round loop of these hangers was suspended on strong glass rods running across the tank and supported by glass shelves. In order to prevent the shifting and rolling of these rods, their ends were provided with short pieces of rubber tubing, resting against the two side walls of the tank and establishing sufficient friction to prevent shifting. A glass beaker filled with water, which was renewed whenever necessary, was provided with three glass atomizers. The water in the beaker was atomized for a period of one minute each day, simultaneously with compressed $SO_2$ gas, compressed $CO_2$ gas and filtered compressed air. The liquid dripping off the baffles, specimens, etc., collected in the front part of the tank and was removed at intervals. An analysis of these drippings showed a content of 63.54 g. of $SO_4$ ions per liter present in the form of free sulfuric acid and soluble sulfates, indicating the extent to which oxidation of the $SO_2$ took place. The tank was covered by a sheet of heavy plate glass which forced the gases to escape through the vent provided for this purpose.

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[1] Chemist and Metallurgist, Metals Coating Co. of America, Philadelphia, Pa.

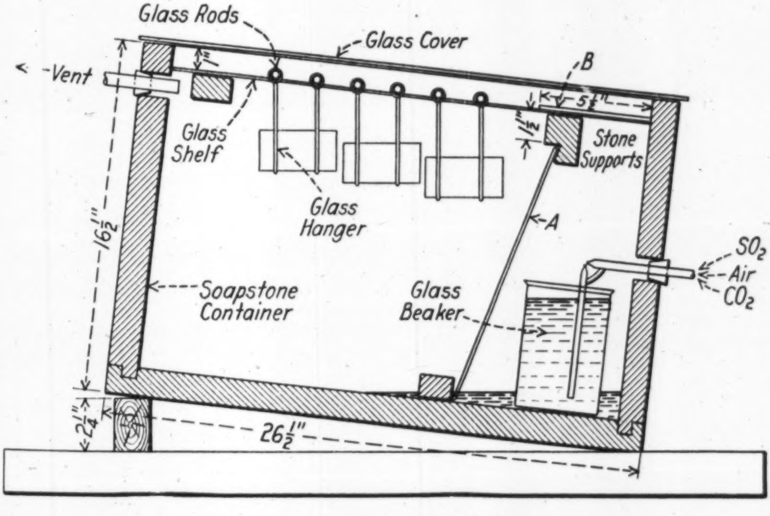

FIG. 1.—Apparatus Used in Tests.

The specimens consisted of blue annealed tank steel sheet ⅛ in. thick in sections of 2 by 4 in. showing the following analysis: carbon 0.13 per cent, manganese 0.60 per cent, sulfur 0.05 per cent, phosphorus 0.05 per cent. The following 23 metals and metal combinations were applied by the metal spraying process: magnesium, zinc, aluminum, cadmium, lead, tin, nickel, monel metal, Nichrome, Enduro KA2, copper, silver, Ascoloy, aluminum over zinc, zinc over aluminum, cadmium over zinc, zinc over cadmium, Nichrome over

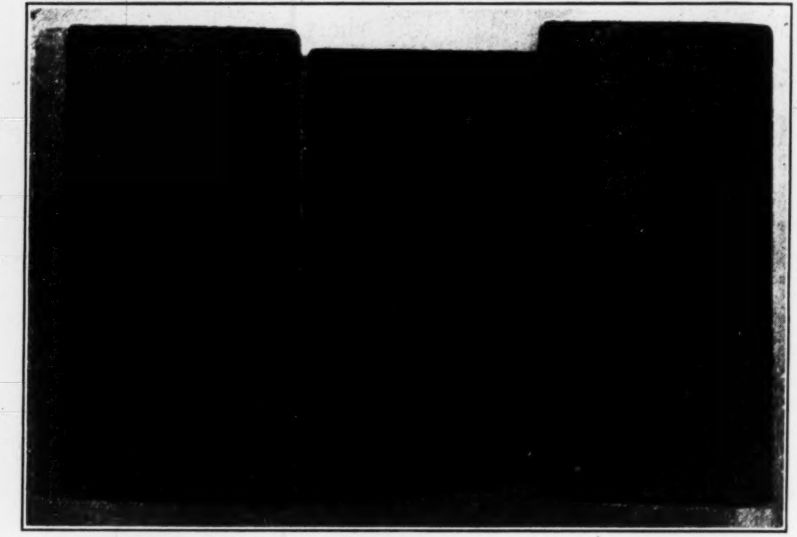

FIG. 2.—Sprayed Molten Lead. Coating darkened, but shows no sign of attack or peeling. Steel completely protected.

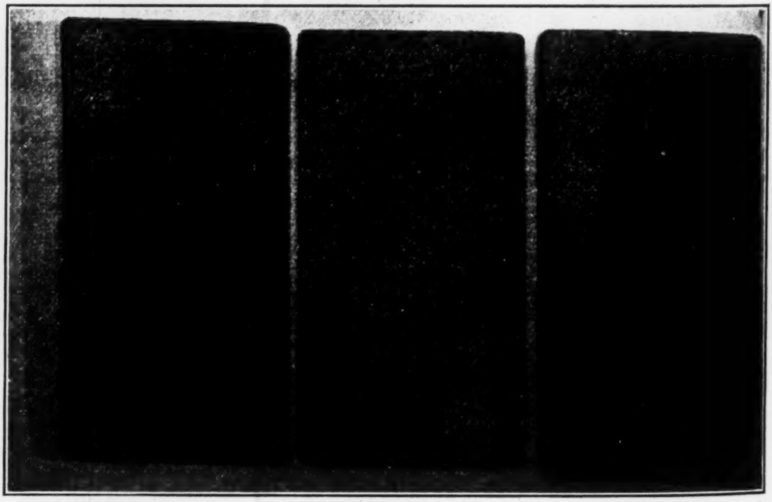

FIG. 3.—Sprayed Molten Lead over Sprayed Molten Enduro KA2. Appearance similar to Fig. 2., with the exception of the plate on the left which shows signs of rust.

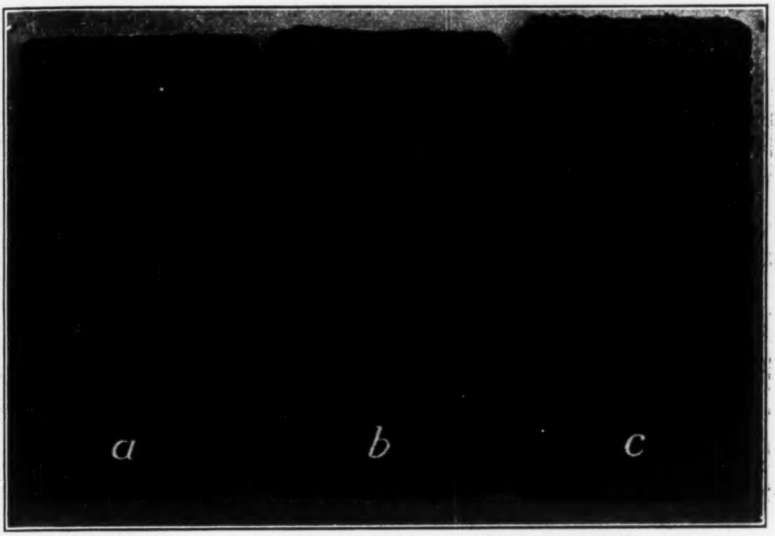

FIG. 4.—(*a*) Sprayed Molten Silver, (*b*) Sprayed Molten Copper and (*c*) Sprayed Molten Tin. Tin coating deteriorated and peeling, silver and copper coatings fairly intact, but peeling. Steel noticeably attacked.

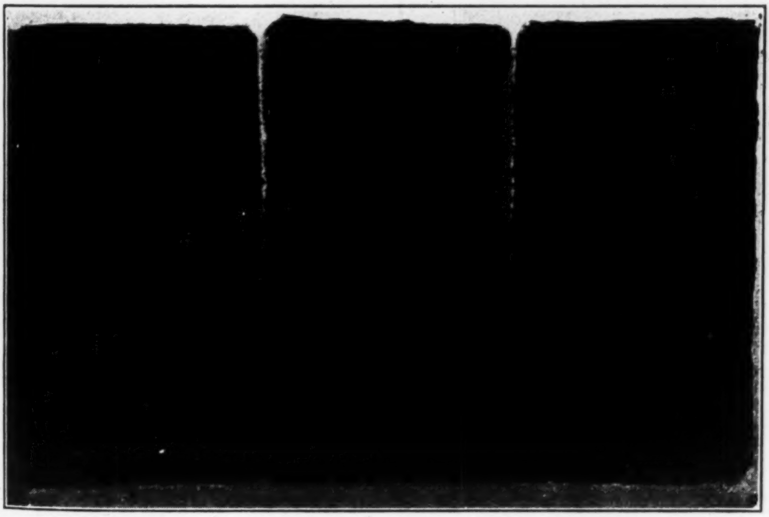

FIG. 5.—Sprayed Molten Monel Metal. Appearance similar to the copper coating in Fig. 4.

aluminum, Enduro KA2 over aluminum, Enduro KA2 over cadmium, lead over Enduro KA2, Enduro KA2 over lead. One set of plates

FIG. 6.—Sprayed Molten Nickel. Coating completely deteriorated, steel severely attacked.

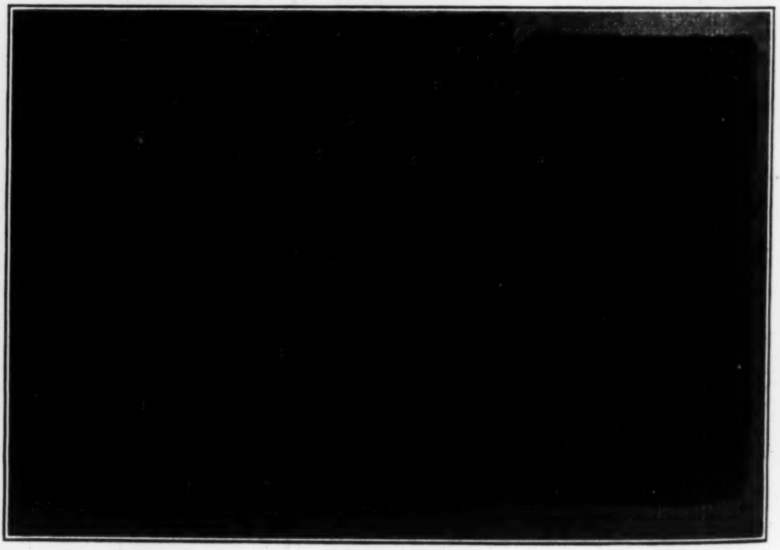

FIG. 7.—Sprayed Molten Nichrome. Coating deteriorated, steel attacked.

was alumetized (the sprayed molten aluminum coating alloyed with the steel by a furnace treatment). The metal wires used for spraying were commercial grades of the best purity obtainable. The thickness of the metal coating in each case was 0.008 in.; in the case of metal combinations it was the sum of the two individual metal coatings

FIG. 8.—Sprayed Molten Aluminum. Coating deteriorated, steel attacked.

FIG. 9.—Alumetizing. Coating deteriorated, steel attacked.

each of which was 0.004 in. thick. Three specimens of each type were prepared and distributed in different sections of the tank. The uniformity of the corrosion on all three specimens within each set indicates strikingly the uniformity obtainable with this type of testing tank. The test was run for a period of exactly one year

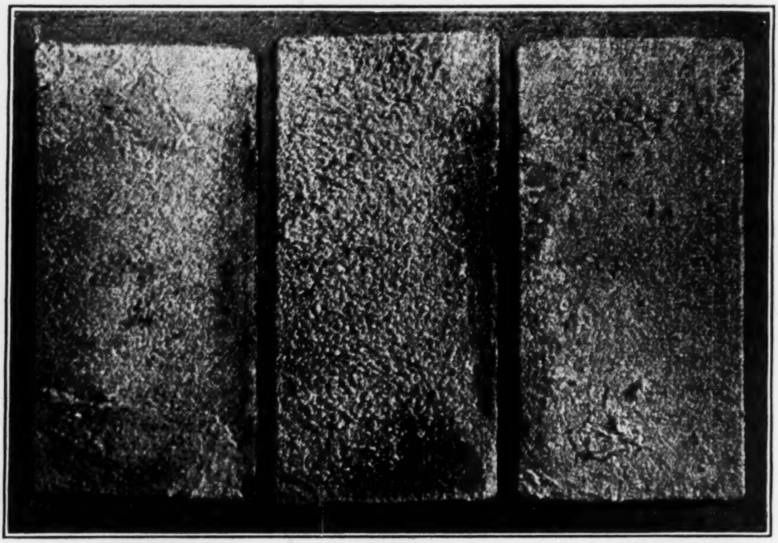

FIG. 10.—Sprayed Molten Zinc. Coating deteriorated, but no visible attack on the steel.

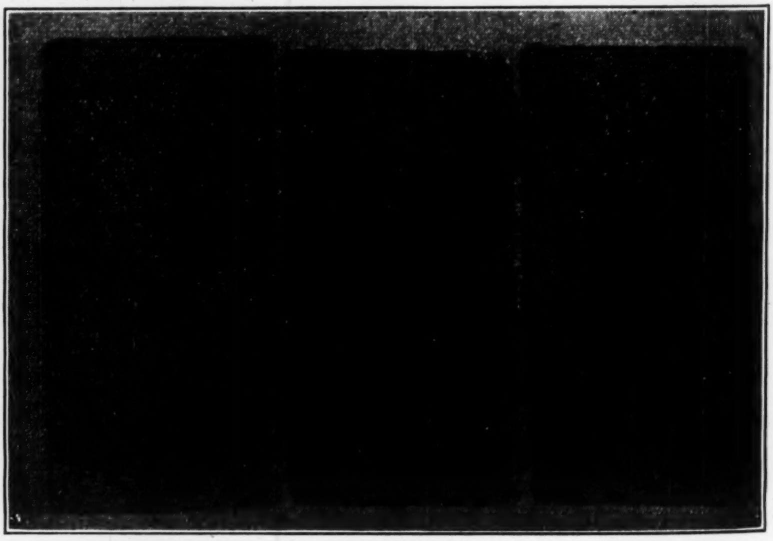

FIG. 11.—Sprayed Molten Cadmium. Coating deteriorated, steel attacked.

The mill scale was removed by blasting with angular hardened steel grit. Blasting with ordinary sand is frequently used and gives a satisfactory bond, except in cases where the coatings are to be

FIG. 12.—Sprayed Molten Aluminum over Sprayed Molten Zinc. Coating completely deteriorated, steel severely attacked.

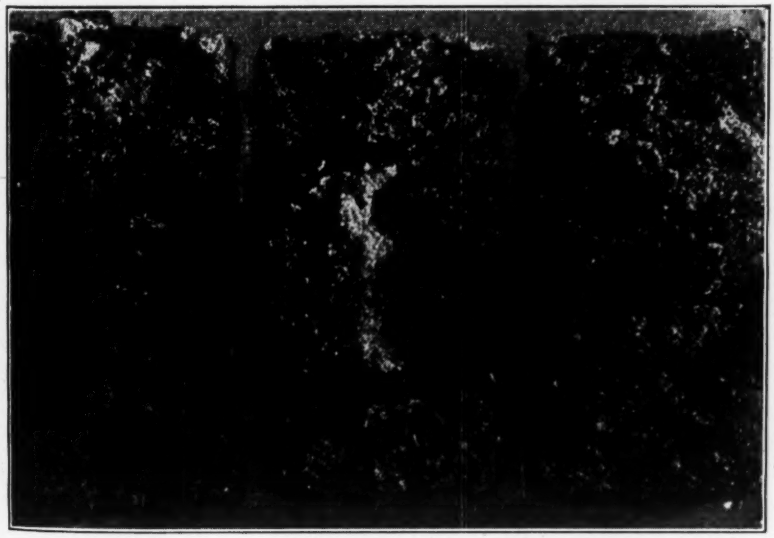

FIG. 13.—Sprayed Molten Cadmium over Sprayed Molten Zinc. Coating completely deteriorated, steel severely attacked.

built up to a greater thickness than usual, or where they are to be subjected to mechanical strains. In such cases the use of steel grit for blasting is absolutely essential. The bond between the sprayed molten metal and the base is a mechanical one and depends upon the interlocking of the sprayed metal particles with the undercuts in the blasted surface. In view of this fact it can readily be seen why steel grit is decidedly preferable to sand.

The evaluation of the test results was done qualitatively only by visual inspection. A service condition which would approximate the conditions within this tank would be found in flue lines, stacks, exhaust fans, etc. It can be presumed, however, that due to the great excess of moisture and $SO_2$ gas prevailing in this tank, the condition is much more severe than under the mentioned service conditions and that the test, therefore, is considerably accelerated. Of the coatings tested, only lead by itself and lead over Enduro KA2 were found to protect the underlying steel. Sprayed molten silver, copper and monel metal coatings were fairly intact but showed a tendency to come loose on the edges which, of course, would destroy their usefulness in actual service where vibrations and heat variations add a mechanical element of destruction. All the other coatings showed more or less severe attack or complete destruction, the corrosion products being always mixtures of sulfites and sulfates. While no definite relation between the test conditions and any specific

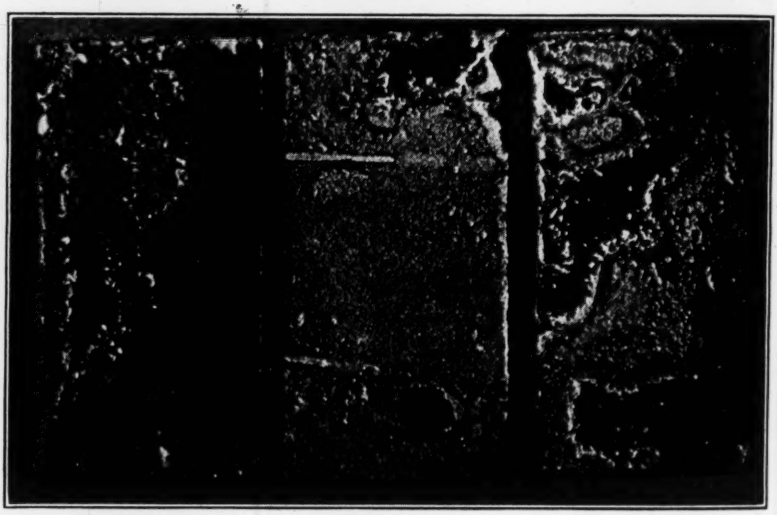

FIG. 14.—Sprayed Molten Zinc over Sprayed Molten Aluminum. Coating deteriorated, steel attacked.

service condition can be established, it can be assumed that sprayed molten lead is far superior to any other metal that may be commercially sprayed as a protection for steel against the combined attack of $SO_2$, $CO_2$, air and moisture.

This conclusion, namely, that lead is the only metal that may be advantageously applied by the metal spraying process for service conditions similar to those accelerated in the apparatus described, may be considered important as it indicates the direction along which future investigations appear to show the greatest promise. It is also confirmed by a considerable number of service tests in which lead was sprayed on fan housings, fan blades and other parts connected with flue and exhaust systems. Most of this work has been done within recent years and the time elapsed since the application of the coatings has been too short to establish data on the average or on the maximum life of these protective coatings. From the observations made so far, however, a minimum service life of two years under average service conditions can safely be assumed.

FIG. 15.—Sprayed Molten Zinc over Sprayed Molten Cadmium. Coating deteriorated, steel attacked.

# SHEAR TESTS FOR CAST IRON

By H. H. Judson[1]

## Synopsis

The service requirements of the castings which are checked by means of the shear test are so severe that a test indicating the strength qualities of the metal in them is highly desirable. The test specimen used, in order to represent the casting, should be typical of it. A piece of the casting itself having been subjected to the same treatment as the casting fulfills this requirement. The specimen used in this test is obtained from a sample which is taken directly from the wall of the casting being examined. The shear apparatus used is nicely made. There are no variables, such as flexing introduced during the actual shearing operation, so true shear values are obtained.

The results of a large number of shear tests and actual bursting tests of cylinders from which the shear specimens were taken show a definite correlation between the shear test values and the quality of the metal in the casting. These same shear tests check very well against carefully made tension tests.

## Introduction

Much has been written on the subject of tests for cast iron and interpretation of these tests. The tension, compressive, transverse, and impact tests are all favorably received and are used by the majority of engineers and foundrymen. The shear test, on the other hand, is little used and evidently less thought of than the others. It has been condemned by most of the authors who have mentioned it on the grounds of unreliability. In Europe where it was first used it is frowned upon by many engineers though favored more by the French than by the British.

The shear test is used in the author's laboratory not because the other types of test are unsatisfactory, but because it fits our particular requirements and conditions The tension test could be used in place of the shear test if it were possible to obtain a specimen large enough for the standard tension test from the casting itself. The shear test has the advantage that an actual piece of the casting in question is used as the specimen. Test data mean little unless supported by information concerning the treatment received by the iron in the test specimen from the time the cupola was charged until the casting cooled down. In the work described the conditions were maintained quite uniform so the test pieces were comparable from casting to casting.

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[1] Foundry Engineer, Goulds Pumps, Inc., Seneca Falls, N. Y.

Most of the data presented on the shear test by those authors who condemn it were obtained on test bars and cast-on coupons. Such specimens do not pass through the same casting and cooling cycle as the castings which they represent. Consider two castings, one of which is a standard $1\frac{1}{4}$-in. round test bar and the other a 2000-lb. high-pressure cylinder. Both of these castings have been poured from the same ladle so the chemical analysis of the iron is the same in both excepting for the proportions of graphitic and combined carbon. The test bar cools very rapidly, resulting in a fine-grained structure throughout. The cylinder is a bulky compact casting and cools very slowly, resulting in a coarser grained structure than that in the bar. The iron in the bar, therefore, is not representative, physically at least, of that in the cylinder. Tests performed on the bar cannot indicate what one may expect from tests on the cylinder because of this dissimilarity in grain structure. The author feels that both the test specimens and the castings they represent should pass through the same program throughout the foundry process. It is on this premise that the statements in favor of the shear test as herein presented are based.

The castings to which this test was applied were cylinders of large reciprocating pumps used to pump crude oil from the oil fields to the refineries. The pumps operate normally under pressures varying from 500 to 1000 lb. per sq. in. Occasionally, however, a pumping station layout is such that pressure oscillations or surges are set up in the pipe lines. These high-pressure waves travel through the oil at a high velocity and may reach a magnitude of several thousand pounds per square inch. Such conditions impose a repeated shock load of high intensity on the pipe line, valves and pump cylinders, and necessitate a high factor of safety and close control of quality in the iron of the cylinders.

Increasing loading demands in this service necessitated the development of higher strength cylinders, so an investigation was begun. Considerable experience had already been gained from pumps in service. Castings from patterns of cylinders in service were made, machined, and tested to destruction. Refinements were made in the design and various so-called semi-steel mixtures were cast in these cylinders which were also tested to destruction. Test bars were poured daily from these heats and were tested transversely and in tension. There was no apparent relation between the test bar results and the results from bursting tests of the cylinders. The bars were so small compared with the cylinders that as castings they had nothing in common except chemical analysis.

The weight of the large castings (Fig. 1) varied from 2000 to 2500 lb., the walls being 2 in. thick for the smaller and $2\frac{1}{4}$ in. for the larger sizes. If a high-steel mixture running low in silicon and carbon was used in these bulky castings they would be gray and machineable, while the test bars would show a white or mottled fracture. If a lower steel mixture running higher in carbon and silicon was poured,

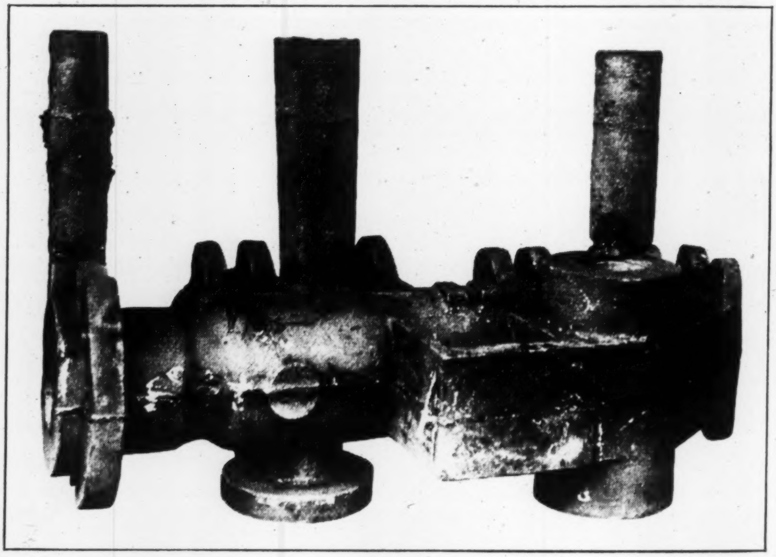

FIG. 1.—Pump Cylinder Casting.

the bars would be excellent but the cylinders would be weak. The actual bursting pressure was the only valuable guide for developing higher-strength mixtures, and by following this guide cylinders were obtained which withstood pressures up to 3500 lb. per sq. in. While this was ample for existing conditions it was thought desirable to obtain a high-test iron which would raise the bursting pressure beyond any possible future demands. As the cost of casting, machining, and testing full-size cylinders was excessive, it was necessary to have some test that would serve as a check on the progress of the investigation and to determine the quality of cylinders being installed in pumps.

## THE SHEAR TEST

During this period, the author's attention was attracted to a paper[1] describing the Fremont shear test for cast iron. The specimens, 0.222 in. in diameter and 0.0387 sq. in. in cross-section (Fig. 2) were obtained from the casting being examined by trepanning with a hollow end mill. They were tested in the Fremont shear machine in either single or double shear. This test seemed to meet our needs, since it employed a specimen which had been an integral part of the casting. A specimen large enough for such a test could be obtained from the wall of each cylinder without damaging it or destroying its usefulness.

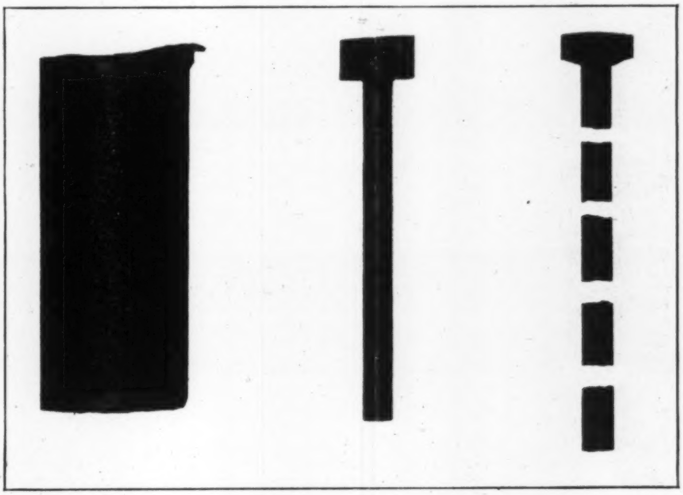

FIG. 2.—Showing Rough Sample, Machined Shear Specimen, and Latter After Testing.

An old testing machine was set up with shear device for testing in single shear and was used for some time. Its lever system had to be changed, so the results obtained were in terms of shear numbers rather than actual shear values, but the data were comparable, which served the purpose. With this apparatus, shear tests were made on samples taken from cylinders which were subsequently tested to failure. At first so little information was available that no conclusions could be drawn, but it was noticed that as the iron was improved and the bursting strengths increased, the shear strength increased also.

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[1] G. K. Elliott, *Transactions*, Am. Foundrymen's Assn., Vol. 34, p. 858 (1926).

The old machine was replaced by an Olsen testing machine with a single shear device of our own design, shown in Fig. 3. The investigation was continued until data were obtained on over one hundred bursting tests of full-size cylinders. The relation between these values in pounds per square inch and the shearing strength of the test specimens is shown in Fig. 4. The highest bursting strengths of 6500 lb. per sq. in. were obtained from an iron low in carbon with a medium silicon content, which, combined with the slow cooling rate of the cylinders produced a pearlitic structure. This iron had a remarkably uniform grain structure throughout the section of the casting and did not chill so readily in the test bar molds. But in spite of this the test bars were of little use as they cooled much too rapidly to develop the pearlitic structure and would not indicate the quality of the cylinder casting.

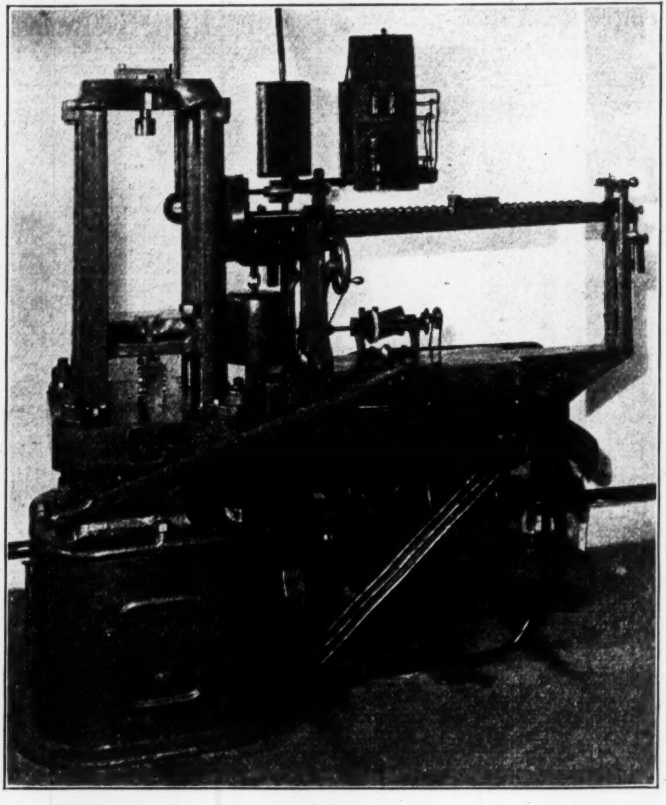

FIG. 3.—Shear Device in Place in Testing Machine.

*Shear Specimens:*

Each cylinder requires a drain plug, the boss for which is shown in Fig. 1. This boss is cast solid and a specimen trepanned from it. Figure 2 shows the rough sample, the finished specimen, and the latter after testing. As the sample is located in the same position on all sizes of cylinders, its cooling rate always bears the same relation to the cooling rate of the rest of the cylinder. Thus a sample is obtained which has gone through the same program as the cylinder from which it was taken and which bears a definite relation to the rest of the casting. It is taken at right angles to the wall of the cylinder and so gives a check on the metal from mold to core. The rough sample taken is turned in a lathe to the exact 0.222-in. diameter and is of sufficient length to make three or four single shear tests, thus eliminating possibilities of error. The specimens are taken from the cylinders in the foundry cleaning room and are immediately machined to size and tested. The chips obtained during machining are used for chemical analysis and the button left on the end of the specimen is used for microscopic analysis. If the test specimens pass the required standards the cylinders are sent to the machine shop, if not they are scrapped.

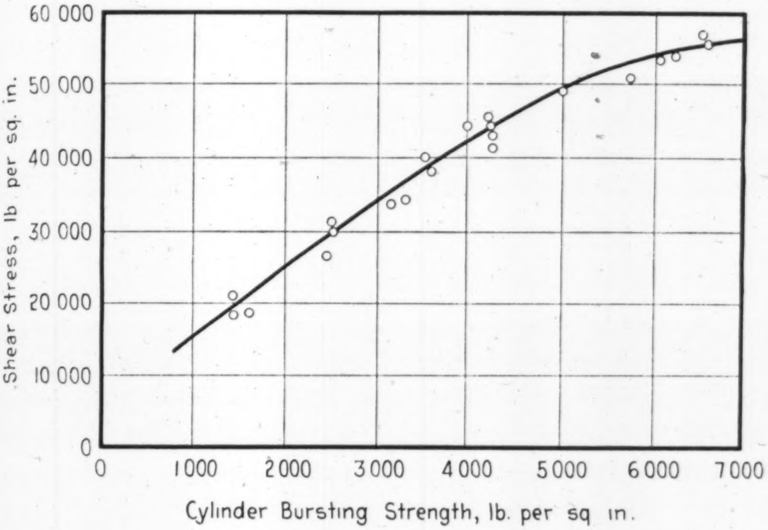

FIG. 4.—Relation Between Bursting Strength of Full-Size Cylinders and Shearing Strength of Specimens.

The shear device, shown in place in the testing machine in Fig. 3, consists of two hardened steel plates, one stationary and one movable.

The stationary plate carries two guides in which the movable plate is free to slide vertically. A hole in the movable plate registers exactly with one in the stationary plate, both being of a diameter to fit the shear test specimen. The movable plate has the hole relieved so that it makes line contact with the specimen where it projects from the stationary plate. The specimen fits the holes so that there is no bending action during shearing; also the movable plate fits the guides nicely to prevent binding.

*Comparison of Shear and Tension Tests:*

A cylinder was broken up after testing to destruction and sections were chosen from different parts. Tension test specimens and shear specimens were made up from these various sections. Care was taken in securing the two kinds of specimens from adjacent portions of the material and parallel to each other to insure that both would contain the same kind of iron as nearly as possible. The results of tests on these specimens are shown in Table I. The tensile strength values are averages of two tests while the shear values are averages of four tests. The data of this table definitely indicate that there is some degree of reliability in the shear test when it is properly conducted and due consideration is given to the manner in which the samples are obtained.

TABLE I.—COMPARISON OF SHEAR AND TENSION TESTS.

| TEST | TENSILE STRENGTH, LB. PER SQ. IN. | SINGLE SHEAR STRENGTH, LB. PER SQ. IN. | RATIO, SHEAR STRENGTH TO TENSILE STRENGTH |
|---|---|---|---|
| No. 1 | 45 800 | 51 400 | 1.12 |
| No. 2 | 45 100 | 49 500 | 1.10 |
| No. 3 | 45 100 | 52 400 | 1.16 |
| No. 4 | 45 800 | 50 000 | 1.09 |
| No. 5 | 42 450 | 49 000 | 1.15 |
| No. 6 | 43 000 | 48 100 | 1.12 |
| No. 7 | 44 450 | 50 400 | 1.13 |
| No. 8 | 47 150 | 55 000 | 1.16 |

## CONCLUSIONS

The curve shown in Fig. 4 is now used to check all line pump cylinders and it is yet to be found wrong. More than 1000 such cylinders have been put in service since the adoption of the shear test as a check and not one field failure has occurred. The test can be relied upon in this work for the following reasons: Because of the ease and promptness with which the specimens are obtained it is possible to keep a very close check on cupola operation, which more than anything else affects the quality of the cylinders. In this work the various sizes of cylinders are all similar in design and are molded, gated, risered and cast the same. The differences in weight and dimensions are so small that the cooling rates of all sizes are quite the same. The same grade of iron is poured into all and the cupola operation during the melting of the iron is held extremely uniform from day to day. Thus a standard set of conditions is maintained which results in a uniform grain structure of the casting.

It is not claimed that the shear test can be used on all materials, neither can it be used to predict tensile strengths in all cases. The work described is one specific application of it and the results obtained justify the confidence placed in it.

# DISCUSSION

Mr. John Shaw[1] (*presented in written form*).—I wish to congratulate Mr. Judson on the careful work he has done. The author has established two points: (1) The futility of using one size test bar for castings of any thickness. This is in line with British thought, which requires that the diameter of the bar should bear a near relation to the thickness of the casting it represents. (2) The author in his Table VI has shown that the ordinary tension test for cast iron is not unreliable as stated by the French school.

I should like to assure Mr. Judson that there has never been any opposition to the use of the shear test as a works routine test,[2] that is, for the consideration of the firms concerned. It has also never been disputed that under the carefully controlled conditions as regards composition, cupola practice, etc., such as obtains in Mr. Judson's case, and in fact in every case where this test has been advocated, that an empirical shear breaking load may be fixed, after trial and error comparisons with the results of the older methods, that will give trustworthy results for that particular composition and practice.

The keen opposition to the "shear" test has been caused by its sponsors' desire to impose this particular test as the dominant test in any proposed international specifications[3] without trying to disprove the very real objections to it. The whole question has been looked at from the producers' or works routine angle and not from the inspecting or designing engineers' viewpoint.

Any international specification for cast iron must comply with at least two fundamental conditions:

1. The test result obtained shall yield a figure that can be converted into tensile strength per square inch (the figure the designer generally uses) or yield some other breaking load figure of known physical properties that he may use to compute the thickness of scantlings necessary for his design.

2. The breaking load figure shall be such that the inspecting engineer can pass with confidence the material as complying with the designer's ideas, so far as the strength of the metal is concerned.

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[1] Southsea, England.

[2] *Proceedings*, Inst. British Foundrymen, p. 553 (1929).

[3] Nicolaus, *Transactions*, Am. Foundrymen's Assn. (1931).

How far will the shear test comply with these conditions when castings from tunnel segments to engine castings are included? In the earlier days it was stated "Shearing is proportional to the transverse. Tensile is equal to shearing. Consequently a shearing test is quite satisfactory for inspection purposes."[1]

Le Thomas stated[2] there was a close relation between shear and tensile strength. The same author later states[3] "It is illusory to look for any relation whatsoever between shear strength and tensile strength. In 1930, at the Liege convention[4] Le Thomas arrived at the same conclusion at which the British Committee arrived after two years' experiments with various compositions, namely, "He deprecated the attempts made to find formulas giving an absolute relation between the results of various testing methods, and said that each test should be taken on its merits." So we now know that the shear result bears no fixed relation to either tensile strength or results of transverse tests, but that it varies with the composition. An examination of data from any source confirms this. On the same machine and under identical conditions except composition, the ratio of shear to tensile strength has varied from 1:1 to 1.9:1. Another vital fact stands out prominently from an examination of results as published in America, England and France: that in all cases the shear result in relation to either tensile or transverse strength is high with the poor material, thus giving a fictitious value to the shear test that would enable poor material to pass that would have been rejected on the present standard tests. In a list published two years ago, the first nine bars 2 by 1 by 36 in. all failed to register the minimum load of 3100 lb. and all material these represented would have failed to be accepted, yet the worst bar of the whole series gave 59,100 lb. on the shear test. The position of both designer and inspector who rely solely on the shear test under these conditions is obvious. It must be remembered that an inspector comes into your works without any knowledge of your analysis or works practice and that he must pass upon your material so far as suitable strength of metal is concerned by the results obtained from one or two simple tests. I should like to call attention to the fact that even in Mr. Judson's graph the shear stress at the lowest strengths is 13.3 times the bursting pressure, while this ratio falls to 8.3 with the highest strength material.

It is claimed, and rightly so, that to cut test specimens from the actual casting is an advantage, especially when the section varies greatly. How much better it is than bars of the same section as the casting, cast attached as a coupon, is open to serious doubt. But are these results not somewhat misleading when stated in terms of stress per square inch? If you take a casting which for reason of imparting rigidity or some other cause has scantlings which vary from 1 to 3 in. in thickness and the tests taken from each position varies from 32,700 lb. in the 1-in. section down to 18,800 lb. in the 3-in. section,[1] does this mean the casting will break under load on the thick weak section? (It may leak under pressure.) A moment's thought will show that the thick section has over five times the strength of the thin strong section. There is a tendency at the present time to attach undue importance to high tensile strength results. This type of metal is entirely unsuitable for many purposes and the writer has met not a few failures that could be traced directly to a very strong metal of this class which had not that small amount of "give" needed in such cases as turbine casings with their slight "breathing" action, or where the casting is subject to a moderate temperature gradient between the outside and the inside. Some of these castings have failed rather from internal stress than undue loading.

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[1] Rongeray, *Proceedings*, Inst. British Foundrymen (1922).

[2] *Transactions*, Am. Foundrymen's Assn. (1926).

[3] Le Thomas, *Proceedings*, Inst. British Foundrymen, p. 500 (1929).

[4] Le Thomas, *The Foundry*, August 1, 1930, p. 95.

MR. HYMAN BORNSTEIN.[2]—This is a very interesting paper, particularly since I believe we need information and data on the shear testing of cast iron.

I note the criticism of the arbitration test bar made by the author and I wonder whether he has made tests on other sizes of test bars which would have more nearly the cooling rate of the casting than the arbitration bar. Why did the author stop with the arbitration test bar? The arbitration bar is not necessarily a control bar. It has been recognized for a long time that, in order to obtain a correlation between test bar and casting, it is necessary to secure a cooling rate in the bar similar to the cooling rate in the casting. Had this been done, would it have been necessary to have gone to the expensive shear method of testing?

Another method of correlation which we have tried out for the past few years is hardness testing. On certain castings we found an excellent correlation between the physical tests on the castings themselves and the Brinell hardness of the casting. Has the author tried out this method of correlation in addition to the shear test?

MR. R. S. MACPHERRAN.[3]—This paper is a very interesting attempt to correlate the test bar and casting. Apparently a close correlation between the shear test and the tensile strength is found. I do not see why this test could not be used for the determination of the actual strength of different parts of the larger castings.

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[1] See Table I, *Foundry Trade Journal*, January 30, 1930, p. 84.

[2] Director of Laboratories, Deere and Co., Moline, Ill.

[3] Chief Chemist, Allis-Chalmers Manufacturing Co., West Allis, Wis.

MR. J. T. MACKENZIE.[1]—The correlation of the shear test and the tensile strength reported in the paper is good. However, they are both subject to the same error; they continue to rise after the iron is too hard for the purpose intended. The French supplement shear tests with determinations of Brinell hardness, which protects them from running over into the hard side on the castings.

However, the test bar of equivalent cooling rate, which in a round bar is of the order of twice the diameter for the thickness of a plate, gives by the deflection an immediate measure of that same property; in other words, one will never get good deflection with an iron after it begins to become mottled for the bending curve stiffens sharply.

I think the ordinary foundryman will find that a suitable size test bar will give him a better, a cheaper and a more rapidly applied criterion of his metal than the shear test, which has to be machined from the casting.

THE CHAIRMAN (*Mr. J. A. Capp*[2]).—If the Chair may be permitted a few remarks, the paper is of a great deal of interest in that it offers something which some of us are inclined to overlook—the sort of data that the man who uses the casting needs. We are inclined to place too much stress on the sort of information that the foundryman needs. Of course he needs it; he has to make the casting and be sure that it is right, but is he always able to assure the engineer that his correlation is such that the engineer may feel that he also is safe?

The fact that the test bar is removed by comparatively simple means, the hollow drill, a tool used regularly in the steel industry, should insure that cost will not be an insuperable objection. Of course it cannot always be used because one cannot always put holes in a casting where one would like to know about it, but just as one of the previous speakers has said, if the relation between tensile strength and shear strength is so nearly constant as has been shown, then that offers the foundryman an opportunity to tell the engineer to whom he sells his castings, just what the casting is good for. It strikes me that the author has shown a possibility that cannot well be overlooked.

MR. H. H. JUDSON[3] (*author's closure by letter*).—Practically all of our experience with the shear test has been in connection with one line of castings and one grade of iron. Our results prove conclusively that it is a reliable test when properly performed. The test is easily made and it is not as expensive as the tension test.

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[1] Chief Chemist, American Cast Iron Pipe Co., Birmingham, Ala.

[2] Engineer, Testing Laboratory, General Electric Co., Schenectady, N. Y.

[3] Foundry Engineer, Goulds Pumps, Inc., Seneca Falls, N. Y.

I do not think that our limited experience with this test warrants a definite reply to Mr. Shaw's objections to its use as an international test. We both agree that it is a good routine shop test and as such it has its merits. There is no reason for condemning the test because the shear varies with the composition of the iron. Neither does the transverse test follow the tension test through variations in analysis.

Mr. Shaw calls attention to the fact that the bursting pressure-shear strength curve does not give the same relations between these values throughout. I am of the opinion that this is due to the design of the cylinder rather than to the shear test because the tensile strengths shown in the table were taken from various parts of one of these cylinders. They check nicely with the shear data. The cylinders fail in tension. In fact, a new design of cylinder showed a low bursting pressure; shear tests made on samples from various parts of the casting, however, were up to standard. A change in design with no change in the composition of the metal brought the bursting pressure up to the point where it was considered amply strong enough.

Replying to Mr. Bornstein, I would state that we started the shear test about six years ago. At the time we started it, there was no thought being given, as far as we knew, to correlating the test bar with the casting as to cooling rate. The shear test has filled our requirements so nicely that we are not considering the use of bars of various diameters. We have not tried to correlate the hardness and shear strength.

I am in accord with Mr. MacPherran's thought that the shear test could be used to determine the strength of various parts of large castings. We have broken up large test castings and tested them in this way with excellent results.

Mr. MacKenzie raises the question of hardness affecting the tensile and shear strengths. The same is true of the transverse test as far as our compositions are concerned. The shear test does increase with the Brinell hardness.

# SYMPOSIUM ON MALLEABLE IRON CASTINGS

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HELD AT JOINT MEETING OF THE
AMERICAN FOUNDRYMEN'S ASSOCIATION
AND THE
AMERICAN SOCIETY FOR TESTING MATERIALS

---

## PREFACE

This Symposium on Malleable Iron Castings is the first of several that have been planned by the American Foundrymen's Association and the American Society for Testing Materials for the purpose of providing the engineering profession with authoritative data in concise form on the properties of metal castings produced by the best present methods of production. In addition to malleable castings it is planned to cover in these symposiums steel castings, both carbon steel and alloy steel, gray iron and other iron castings, and castings of various non-ferrous metals.

The undersigned committee was appointed by the two societies to compile, critically analyze and assemble for publication, data on the properties of malleable iron castings useful to engineers and metallurgists engaged in designing and specifying such castings. Many sources have been consulted and every effort made to make the compilation as complete and authoritative as possible.

The following companies supplied data for one or more sections of the Symposium, and their cooperation is gratefully acknowledged:

American Chain Co., L. C. Wilson
Atchison, Topeka and Santa Fe Railway System, The, E. E. Chapman
Belle City Malleable Iron Co., D. P. Spencer
Cadillac Malleable Iron Co., R. J. Teetor
Chicago and North Western Railway Co., H. D. Browne
Chicago Railway Equipment Co.
Crane Co., L. W. Spring
Dayton Malleable Iron Co., The, John Haswell
Detroit Testing Laboratory, The, W. P. Putnam
Erie Malleable Iron Co., W. C. Missimer
Fort Pitt Malleable Iron Co., L. J. Kelley
General Electric Co., I. R. Valentine
Grabler Manufacturing Co., F. E. McCabe
Jefferson Union Co., R. E. Bryant
Malleable Iron Fittings Co., A. H. Jameson

Malleable Iron Research Institute, Enrique Touceda and H. H. Lind
National Malleable and Steel Castings Co., H. A. Schwartz
Ohio Brass Co., F. L. Wolf
Rockford Northwestern Malleable Corp., D. P. Forbes
Ross-Meehan Foundries, F. M. Robbins
Stockham Pipe and Fittings Co., E. K. Smith
Union Malleable Iron Co., The, E. A. Gullberg
Wilmington Malleable Iron Works
York Ice Machinery Corp., C. H. Ehrenfeld

In at least two instances, the data supplied by companies in the above list included results of tests from other companies. While the names of these latter companies are for the most part not known, the committee's thanks are due them even though they remain anonymous.

The Committee, on behalf of the two sponsor societies, expresses its thanks to the malleable industry and the Malleable Iron Research Institute for their whole-hearted support of the Symposium. Special acknowledgment is made to the authors of the two technical papers, and to Messrs. Putnam, Schwartz and Touceda, members of the committee, for many helpful contributions to the text of the Symposium.

Respectfully submitted,

| | |
|---|---|
| C. L. WARWICK, *Chairman* | H. A. SCHWARTZ |
| R. E. KENNEDY | ENRIQUE TOUCEDA |
| W. P. PUTNAM | I. R. VALENTINE |
| F. M. ROBBINS | O. B. ZIMMERMAN |

*Committee on Symposium*

# SYMPOSIUM ON MALLEABLE IRON CASTINGS

## I. INTRODUCTION

The importance of malleable iron as a material of engineering emphasizes the need of accurate, reliable information upon its manufacture, its properties and other facts of value to the user of the material. It is the primary purpose of this Symposium to meet this need.

Malleable iron as considered in this Symposium refers to the product produced by the annealing or graphitization of "white iron" castings, in which all carbon should be present in the combined form, in such manner that the final structure of the malleable casting consists of ferrite and free carbon in the form of "temper" carbon, with substantially no combined carbon in the form of free cementite or pearlite present. The Symposium does not cover the type of malleable iron produced in European countries in which a more or less completely decarburized structure containing no temper carbon is sought; nor does it include castings produced by modifications of the original annealing cycle of a character to retain combined carbon in some desired metallographic form.

It is desirable to make clear that the terms "free carbon," "graphitic carbon" and "graphite" are synonymous and have been used interchangeably in the Symposium. Free carbon formed during the annealing or graphitizing process in the production of malleable iron from the original white iron is termed "temper carbon," which under ordinary conditions is nodular in form. "Primary graphite" is the specific variety of graphite which forms during freezing and as used in the Symposium refers to the flake-like form of graphite.

Considering methods of manufacture, malleable iron falls broadly into two classes: that produced by the air-furnace, open-hearth or electric-furnace process, or one of the duplexing processes, and that produced by the cupola process. By far the greater amount falls in the first class, and the Symposium deals principally with that product. The description of process of manufacture given in Section II applies to this class, although the discussion of metallurgical features of the process is generally applicable. The cupola process is briefly described in Section VI.

The air-furnace and similar processes are capable of producing a variety of grades of malleable iron castings, of varying mechanical properties suited to a variety of uses. The Standard Specifications for Malleable Castings (A.S.T.M. Designation: A 47–30) of the American Society for Testing Materials[1] are representative of probably the majority of malleable castings as produced today. Accordingly, it has not seemed necessary to attempt any subdivision of the product by ranges of strength or other properties in presenting the data given in Sections III and IV. However, there are uses of malleable iron, for example, in certain railroad equipment, where high strength and ductility are major requirements, and emphasis upon higher strength malleable iron has justified the presentation in Section V of tensile properties of such iron produced to meet recent specifications. On the other hand, in some machine parts and automotive castings the requirements for strength and ductility are not so severe and a lower strength product is used with entire safety.

Cupola malleable iron is used principally in the manufacture of pipe fittings. It is usually less ductile than iron made by the air-furnace or similar process. Data on tensile properties are given in Section VI; no data on other mechanical properties of cupola malleable iron were available.

The data presented in Sections III, IV, V and VI of the Symposium, and in particular the average values given for the various properties of malleable iron discussed therein, have been critically analyzed by the Symposium Committee, reviewed by the appropriate committees of the American Foundrymen's Association and the American Society for Testing Materials, and are offered as authoritative, trustworthy data, representative of present-day production of malleable iron. The data on physical constants, mechanical properties and electrical and magnetic properties given in Section III were obtained from about twenty contributing sources, including many of the leading producing companies and the Malleable Iron Research Institute. Since the tension test is the specified acceptance test for the product, upwards of twenty thousand tests for tensile strength, yield point and elongation were made available to the committee, and form the basis of the average values reported for those properties. While fewer tests have been made for other mechanical properties, such as as modulus of elasticity, shearing strength, impact strength and hardness, for physical constants such as specific gravity, thermal expansion and specific heat, and for electrical and magnetic properties, they are known to have been made carefully by competent authorities and the average values reported are considered reliable and representative of the product.

The article on microstructure shows normal structure and typical defective conditions that may develop through some fault in manufacture. No effort has been made to include conditions not met in commercial practice.

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[1] 1930 Book of A.S.T.M. Standards, Part I, p. 491.

Supplementary information on and a discussion of tensile properties are offered in Section IV. Data on the range of tensile strength, yield point and elongation as observed in two groups of tests, one representing 5000 bars and the other about 650 bars, are given, with a brief statistical analysis of the data in the first group. The unique relationship of elongation and tensile strength of malleable iron is discussed in this section, as well as the effect of machining and of cross-sectional area upon strength.

All of the strength data in the Symposium have been obtained on test bars, either separately cast (the usual custom) or attached to the casting, as distinguished from specimens cut from some portion of the casting. In common with other types of castings, the strength of a malleable iron casting as compared with that of the test bar cast from the same metal is dependent upon a number of factors inherent in foundry operations, such as the design of the casting, the thickness of various sections, methods of gating and venting, rate of cooling and soundness of metal.

In this connection, it is desired to emphasize the importance of cooperation between the engineer and the foundry, in order that the principles of good foundry practice may be suitably observed in the design of malleable castings. Section VII is devoted to a discussion of this subject.

As a matter of interesting record, there has been included as Section VIII of the Symposium a brief résumé of current specifications for malleable iron castings, in which have been listed a number of the specifications promulgated by leading consumers, by manufacturers and by technical societies, with a tabulation of the physical properties specified. The Committee has not attempted to include all known specifications, but the list is sufficiently complete and representative to give a clear picture of the present status of specifications for this product.

## TECHNICAL PAPERS

There are but few quantitative data available respecting two of the most important properties of malleable iron, namely, its machineability and its resistance to corrosion. Hence, except for a brief paragraph or two on machineability in Section III, these two subjects are not covered in the Committee's compilation of data. Instead two technical papers have been secured which are appended.

As its title implies, the paper by O. W. Boston on "An Investigation of Methods to Determine the Machineability of Malleable Iron

Castings" describes the methods used by this investigator which have been applied to similar work on other metals. Data on machineability of malleable iron are presented, which it is hoped will be supplemented by further studies that are being made. This paper is a distinct contribution to the subject and merits careful study.

The paper on "Corrosion of Malleable Iron" by F. L. Wolf and L. A. Meisse describes the studies of pioneer workers in this field of research, which has an important bearing on the uses of the product. It is hoped that this paper will stimulate other investigations where malleable iron is subjected to severe corrosive conditions.

## USES OF MALLEABLE CASTINGS

Where strength, ductility, machineability and resistance to shock are important considerations, malleable iron castings find a wide field of industrial applications. A general classification of industries using malleable iron castings, and a list of parts in certain industries, was furnished to the committee by the Malleable Iron Research Institute. While too long to publish in full, the following abstracts therefrom give an excellent idea of the range of uses of the product:

### CLASSIFICATION OF INDUSTRIES, WITH CONDENSED LIST OF PRODUCTS, IN WHICH MALLEABLE CASTINGS ARE USED.

*Agricultural Implements*

Plows, tractors, harrows, reapers, mowers, binders, cultivators, rakes, spreaders, dairy and poultry equipment, presses, tools, pumps, wagons, fence parts.

*Automobiles*

Parts for frame, wheels, springs, brakes, motor, transmission, axles, steering gear, body, accessories.

*Boilers, Tanks and Engines*

Boilers, tanks, fittings, engine parts, outboard motors, diesel engines.

*Building Equipment*

Hardware for windows, doors, garage equipment, awning hardware.

*Conveyor and Elevator Equipment*

Chains, buckets, pulleys, rollers, cranes, hoists, fittings.

*Electrical and Industrial Power Equipment*

Motor and generator parts, pumps, stokers, electric locomotives and tractors, steam specialities, outlet and switch boxes.

*Hardware and Small Tools*

Pneumatic and portable tools, miscellaneous tools, saddlery, hardware, table and kitchen utensils.

*Household Appliances*

Stoves, sewing machines, refrigerators, washing and ironing machines, vacuum cleaners, dishwashing machines, oil burners, electric fans and toasters, radios.

*Machine Tools*

Lathes, planers, shapers, grinders, screw machines, gear cutters, drills.

*Machinery for Special Uses*

Textile, cement, rubber, shoe, mining and quarrying, grinding, forging, foundry, bakery, wood-working, bottling, ice, laundry industries.

*Marine Equipment*

Anchors, chains, capstans, fastenings, towing bits, hardware.

*Metal Furniture and Fixtures*

Stoves, beds, desks, filing cabinets, shelving, hotel supply equipment.

*Municipal, State and Public Service*

Manhole covers, posts, guard rail equipment, highway markers, transmission and telephone line fittings, electric railway fittings.

*Pipe Fittings and Plumbing Supplies*

Elbows, unions, reducers, flanges, valves, bolts, nuts.

*Railroads*

Wide variety of parts for construction of locomotives, freight and passenger cars; guard rails, miscellaneous.

*Road and Contractor's Machinery*

Rollers, excavators, cranes, hoists, tractors, graders, scarifiers, mixers, pavers.

*Toys and Specialties*

Sleds, wagons, automobiles, bicycles, carriages, gun parts.

According to the Department of Commerce, there are about 150 plants in the industry, with an estimated annual capacity of 1,350,000 tons.

## II. MANUFACTURE OF MALLEABLE IRON CASTINGS

The usual procedure followed in the manufacture of malleable iron castings involves the following steps:

*Melting.*—Producing a hard brittle white iron from a charge of pig iron and scrap.

*Molding.*—Preparing molds, in green (damp) sand from patterns, in which to pour the melted metal to give "hard-iron" castings.

*Pouring or Casting.*—Transfer of metal from the furnace and pouring in molds, cooling and removing the hard-iron castings from the sand molds.

*Hard Iron Trimming, Cleaning and Inspection.*—Removing sprues and fins from the hard-iron castings, cleaning by tumbling or sand blasting, and discarding defectives on inspection.

*Packing for Annealing.*—Placing castings in annealing pots.

*Annealing or Malleablizing.*—Converting hard white iron castings to black-fracture, soft machineable castings.

*Sorting, Cleaning and Inspection of Malleable Castings.*

### METALLURGICAL FEATURES

The malleable iron metallurgical process is conducted in two steps. The first pertains to the making of what is known in the foundry as the "hard-iron" castings, white in fracture, in which all carbon should be present in the combined form. The second step consists in converting, through a proper heat-treating cycle, these hard and exceedingly brittle castings into an easily machineable product, which while soft and ductile is surprisingly tough and strong, and in which all the carbon is present as free carbon or graphite. While this step is known ordinarily as the "annealing process," the term is not fully descriptive in that annealing is only an incidental part of what is accomplished. In this description, the terms "annealing," "heat treatment" and "malleablizing" are used synonymously.

*The First Step—Manufacture of White Iron Castings:*

To understand the significance of the first step, it is necessary to consider briefly the relation of chemical composition and structure of cast iron.

Cast iron is a complex alloy of iron and carbon in which the carbon can be present as carbide of iron and free carbon or graphite. The latter makes the fracture gray or black; its absence permits a white structure. Cast iron can be completely white in fracture at one extreme end of a series, or intensely black at the other; the complete series will shade from white into slightly mottled, heavily mottled, very light gray, darker grays to black.

Neglecting the influence of sulfur, which acts to retain carbon in the combined form, a white-fracture iron will normally be so low in silicon that during or after solidification no carbon will be precipitated as graphite. Consequently, all the carbon will be chemically combined with some of the iron to form carbide of iron, $Fe_3C$, a definite chemical and structural constituent of great hardness and brittleness. The presence of a slightly higher silicon content than would be contained normally in white iron will serve to precipitate as graphite a small proportion of the carbon, producing a slightly mottled fracture. With a somewhat higher silicon a more completely mottled condition will be found. With further increase in silicon, the grayish fracture will develop into one that is black. As the graphite increases in amount, the iron becomes softer and weaker, until a point is reached where the silicon content is so high that all of the carbon is present as graphite.

With a knowledge of these facts, the manufacturer of gray-iron castings has at his disposal a means of making hard or soft castings, or castings of intermediate hardness as may be specified. The malleable iron founder, on the other hand, makes use of these metallurgical principles with the sole purpose of securing a casting in which the carbon is entirely in the combined state; otherwise the castings when malleablized will be of inferior quality.

*Furnace Charge.*—The furnace charge from which malleable castings are to be made usually consists of a mixture of 30 to 40 per cent of pig iron, 45 to 50 per cent of sprues from a previous heat, and the balance of malleable scrap, purchased or resulting from defective castings at the plant. At times a small percentage of steel scrap is used, mainly for carbon adjustment.

Considering the common elements only, the average composition of the charge will be about as follows:

| | |
|---|---|
| Carbon | 2.50 to 3.00 per cent |
| Silicon | 1.20 to 1.30 " |
| Manganese | 0.55 to 0.65 " |
| Phosphorus | under 0.18 " |
| Sulfur | under 0.065 " |

The variations depend upon whether the plant is running on a high or low tensile strength product. The elements shown in the above typical analysis of a charge must be restricted within certain limits, in order that the castings may be properly and completely malleablized. Unless the elements are present in proper proportion the finished product will be defective.

*The Second Step—Graphitization by Heat-Treating Process:*

The graphitization (malleablizing) of the castings is accomplished by means of a suitable heat treatment, which upon completion will have converted the hard white iron (see photomicrograph, Fig. 1) into a ductile, strong, tough and easily machineable product, in which no hard constituent will be present (see photomicrograph, Fig. 2.)

In white iron suitable for graphitization there are but two elemental constituents, namely, the relatively soft ferrite (pure iron) and the extremely hard, brittle cementite (carbide of iron, $Fe_3C$), although a part of the latter is mechanically mixed with all of the ferrite to form pearlite (12 parts cementite and 88 parts ferrite) of intermediate hardness, the balance existing as free or excess cementite. When such iron is heated to just above the critical temperature, about 1400° F., the pearlite is transformed into austenite, the cementite remaining unchanged. Austenite is a constituent of indefinite composition with respect to carbon, capable of dissolving carbon in amounts proportional to its temperature. Accordingly, if now the temperature is raised to around 1550 to 1600° F., the austenite will absorb as much carbon from the cementite as it can hold at that temperature. If this temperature is maintained, the austenite gives up some of the absorbed carbon, which is precipitated in the form of "temper" carbon. As this precipitation takes place, the austenite

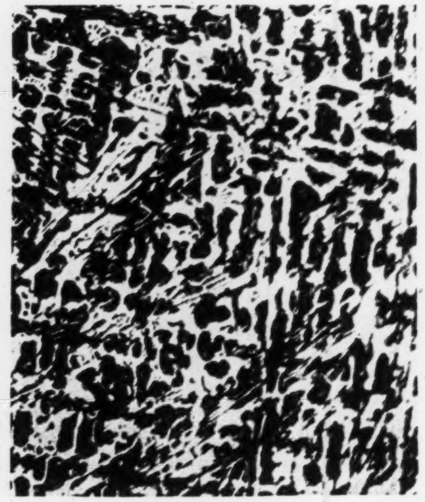

FIG. 1.—Structure of Air-Furnace White Iron that Yields a Satisfactory Product (× 100).

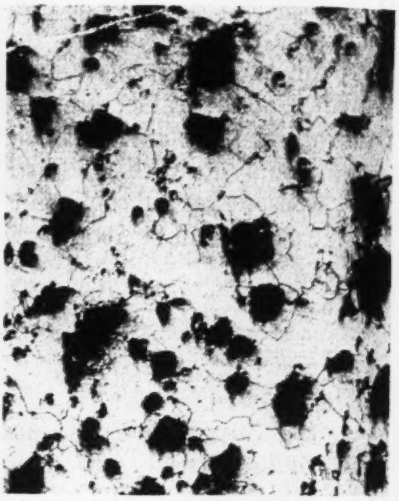

FIG. 2.—Normal Structure of Malleable Iron (× 100).

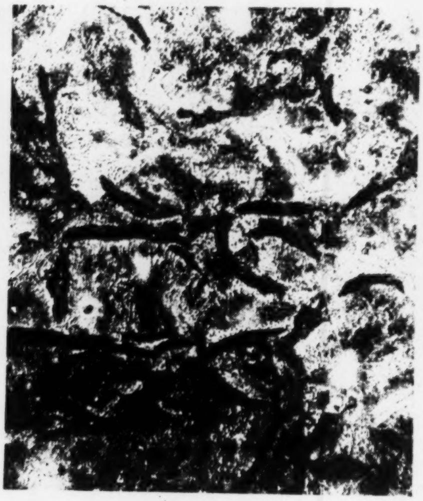

FIG. 3.—Structure of Gray Iron of High Strength (× 100).

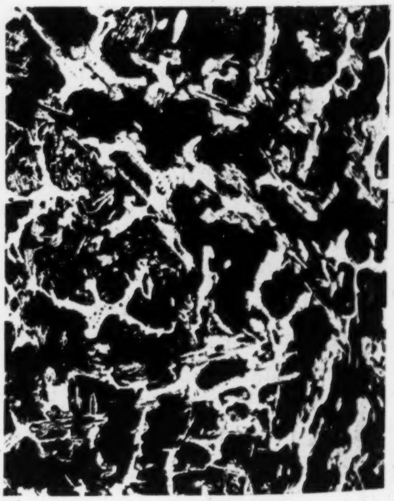

FIG. 4.—Structure of Air-Furnace White Iron of Too High a Carbon Content to Produce the Best Quality of Product (× 100).

All Specimens Etched with 10-per-cent Nital.

absorbs more carbon from the cementite and the cycle is repeated during the "soaking" period until all the carbon in the cementite has been dissolved. At this stage some carbon has been precipitated and the remainder is in the austenite. Then the temperature is very slowly lowered and more temper carbon is precipitated from the austenite. At the critical temperature, however, the austenite still contains the amount of carbon originally present in the pearlite of the white iron. To prevent reversion to the relatively hard pearlite in passing through the critical temperature, and to effect complete precipitation of the carbon, a very slow rate of cooling down to about 1275° F. must be maintained, which permits the breaking up of the austenite into ferrite and temper carbon, the final constituents of malleable iron.

In brief, the malleablizing process has converted hard, brittle white iron with a structure of pearlite and cementite into malleable iron, with a structure of relatively soft ferrite and temper carbon. This structure is illustrated in Fig. 2. (Compare also Fig. 14, in the discussion of Microstructure.[1])

Coincident with the changes that take place in the structure of the iron above its critical temperature during this heat treatment, a certain amount of carbon gradually is eliminated from the immediate surface of the casting directly exposed to the oxidizing furnace atmosphere, resulting in a decarburized surface or "skin."

*Structure Making for Strength and Uniformity.*—For a more complete understanding of why malleable iron is so uniform in structure, it is essential to make certain comparisons between the structure of ordinary gray iron and white cast iron. The photomicrograph, Fig. 3, shows the structure of a piece taken from a high strength gray-iron casting. It will be seen that the graphite exists in the form of flakes that vary in shape and size, and are not uniformly distributed throughout the section. In the photomicrograph, Fig. 4, is shown the structure of air-furnace white iron of a carbon content higher than should be present in a good hard-iron casting. In spite of the fact that in this case the pearlite and free cementite are present in rather massive form, it will be noted that they are quite uniformly distributed in respect to each other. The white iron structure shown in Fig. 1 is best suited for castings that will yield good tensile properties.

## DESCRIPTION OF MANUFACTURING PROCESSES

*Melting:*

In the majority of cases the mixture is melted in a reverberatory type of furnace commonly known as an air furnace, which in most instances is hand-fired, although pulverized coal or oil is now used by a large number of plants. In a few instances, melting is accomplished in an open-hearth furnace instead of an air furnace, admitting of better control. Where the cost of current is not prohibitive, a limited number of plants make use of the electric furnace.[1]

---

[1] See p 350.

There have appeared recently designs of small rotary furnaces of two to ten tons capacity, one of which recently has been installed in this country.

There are also several installations where duplexing is carried on, which serves to provide a continuous supply of metal. The metal is melted in a cupola and refined and superheated in the electric furnace or in the air furance. The pipe fittings industry makes use very largely of metal melted by the direct cupola process.

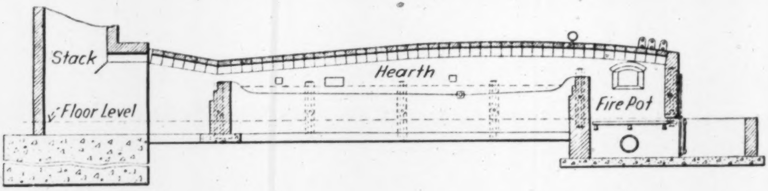

FIG. 5.—Longitudinal Section Through Air Furnace.

*Air Furnace Melting Process.*—As the air furnace is the common type in use for melting, a brief description of the process is given.[2] The air furnace consists of three main parts: a fire pot, hearth and stack. A longitudinal section through a typical furnace is shown in Fig. 5. The roof consists of a series of rather flatly arched bungs placed side by side and held in place by adjustable metal frames which can be removed as occasion requires. The side-wall fire brick are laid against heavy cast-iron plates which form the outside of the furnace. The furnace bottom is made up with a refractory lake or sea sand, or other suitable material. Uusually the metal is drained from a tap hole with pouring spout located at the lowest point of the hearth. Openings are provided to permit manipulation of the stock as it is melting down and skimming off the slag as occasion requires.

With the furnace at a high temperature from a previous heat the furnace charge is dropped into the hearth through a temporary

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[1] Attention is directed to the fact that in this Section of the Symposium cupola process iron is not being considered. For information on that process, see Section VI, p. 370.

[2] The reader is referred to the standard texts and treatises on metallurgical processes for detailed information on design and operation of furnaces and other equipment used in the manufacture of malleable iron.

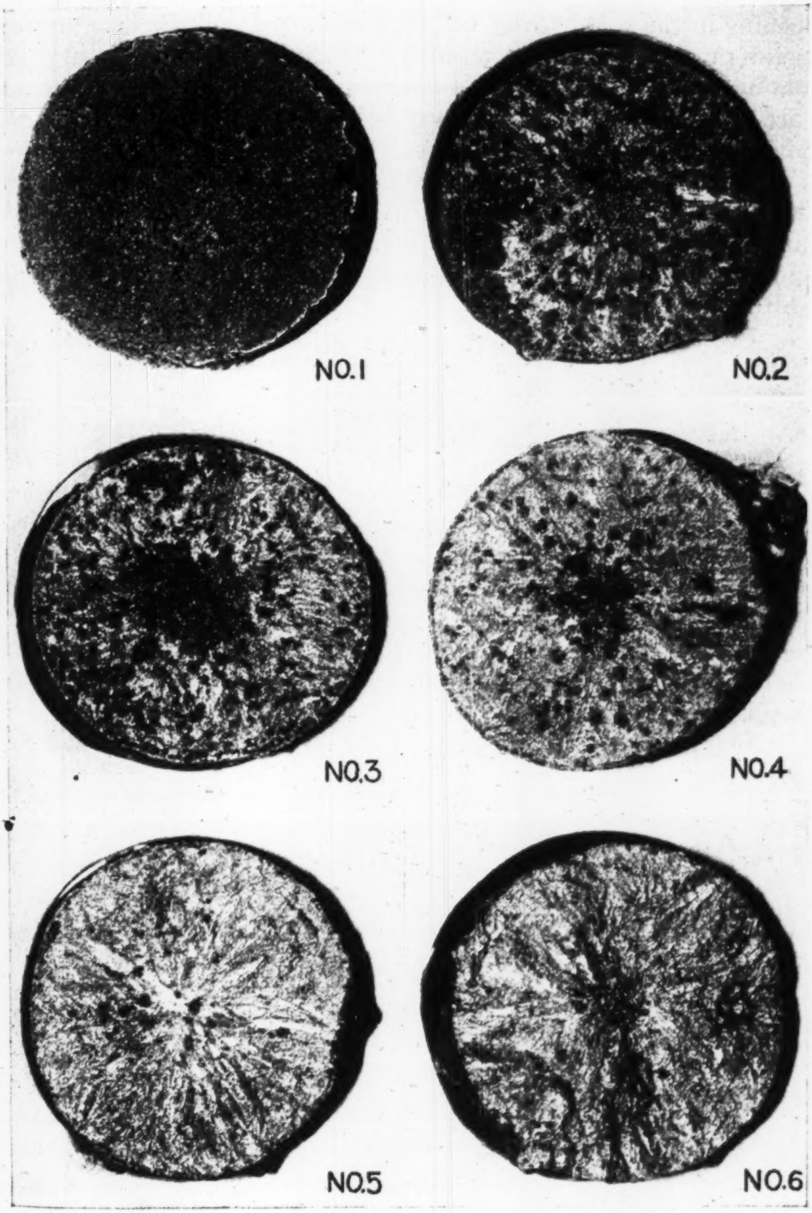

FIG. 6.—Test Sprues Taken at Half-Hour Intervals During Melting of Heat of Malleable Iron.

**No. 1 shows presence of graphite when charge is just melted.**
**No. 6 shows complete absence of graphite when metal is ready to pour.**

opening in the roof. After the air is admitted and the heat started, melting proceeds with fair rapidity. If the heat is of a size approximating 15 tons, about 3½ to 4 hours will be required before the furnace will be ready to tap. Over half the time is taken in melting down the charge to a consistency that will permit the slag to free itself from the metal. The remainder of the time is required to get the metal to a temperature sufficiently high to prevent the misrunning of the castings and to allow for the elimination of enough silicon and carbon to make sure that the fracture of the castings will be white.

*Test Sprues.*—As a guide to the rate of elimination of silicon and carbon after the charge has just completely melted, the furnace-man from time to time withdraws some molten metal, casts it in the form of a "test sprue" and examines the fracture. The fracture of a test sprue cast when the charge was just completely melted would be found to be dark gray, indicating the presence of graphite in large amount. Successive tests at half-hour intervals would show a heavily mottled condition, a still less amount of mottle and then rather quickly a gradual clearing up of the metal; until finally no trace of graphite would be visible in the test sprue fracture. At this point, if the initial silicon in the mixture has been adjusted properly, the bath should be sufficiently fluid to run the thinnest work on the molding floors.

In Fig. 6 are shown fractures of test sprues taken at half-hour intervals from the time when the melting of the charge was just completed to the time of tapping the furnace. The sprue marked No. 1 is the start of the series.

Through careful observations, the approach to adverse conditions can be recognized in time to adopt corrective measures. Through the proper use of ferro-silicon, silico-spiegel, ferro-manganese and petroleum coke, any deficiency in either silicon, manganese or carbon can be remedied.

### *Pouring and Cleaning:*

After the molds have been poured, the castings are shaken out, the sprues and gates broken off, and the castings cleaned in tumbling barrels or by sand blast. From the cleaning apparatus they are taken to the trimmers and inspectors, the former to remove fins and similar surface imperfections from the castings, and the latter to discard such castings as show an imperfection of any kind.

If test specimens representing the melt are called for, as for example in the A.S.T.M. Specifications for Malleable Castings

(A.S.T.M. Designation: A 47),[1] they are poured at the same time as the castings and suitably marked for identification with the melt.

*Annealing or Malleablizing:*

Following these operations the castings are taken to the annealing room, where they are to undergo the second step in the process, that is, malleablizing. The following description of the procedure applies to the periodic type of annealing oven.

*Packing.*—The castings are placed in cast-iron pots or rings and are surrounded by a packing material which may consist of squeezer slag, crushed air-furnace or blast-furnace slag, used alone or mixed with mill scale or other form of iron oxide. In many cases sand is used. Whatever the packing material, it should be sufficiently refractory not to fuse on the castings at annealing temperatures. When one pot has been filled, another pot is superimposed on it and filled in a similar manner; followed by a third and fourth. These together constitute what is known as a stand. Over the top of the stand are placed flat pieces of worn-out rings; and over this is plastered clay to prevent the access of oven gases to the castings. The joints between the rings likewise are luted with clay.

The principal object of the packing is to make as nearly as possible a solid mass of the contents of the stands, to the end that the castings will be supported in a manner that will prevent warpage.

*Annealing Period.*—The stands are placed in the oven in rows with suitable clearance. Any test specimens cast to represent the castings of a given melt are suitably packed and placed in the oven with those castings. A rather slow firing is started and the temperature is increased at such rate that in some two days' time the castings are at the annealing temperature, which in some cases will be 1550° F. and in others about 1600° F. In either case this temperature is maintained for from 48 to 60 hours. At the end of this period, the temperature is lowered at a rate of not more than 8 to 10° F. per hour until the temperature has fallen to around 1275° F., after which the oven doors can be opened and the stands removed, and the castings shaken out as soon as their temperature permits.

*Types of Annealing Furnace.*—The periodic type of annealing oven, to which the above description applies, predominates in general usage. In later years there has been a marked improvement in the annealing furnaces. Ovens of the periodic type for the most part are now thoroughly insulated and run under pyrometric control. These ovens are of rectangular box form with a fire pot at one end on the outside, the products of combustion passing from it to within the oven proper over a communicating bridge wall. While most of the ovens are hand-fired, oil, natural gas and powdered coal also are used. The capacity varies from about 25 to 45 tons of castings charged, and the total time required for the malleablizing process will average about seven days.

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[1] 1930 Book of A.S.T.M. Standards, Part I, p. 491.

One of the best ovens in use is the muffle oven, which consists of an insulated rectangular brick shell supported on the outside by steel plates, buck stays, etc. Within this shell a fire-brick muffle is constructed with a space between it and the brick shell sufficient for proper combustion of the fuel used, and so arranged with flues as to enable the flame to heat the entire surface of the muffle. In most cases oil or natural gas is used. Since the products of combustion do not come in direct contact with the castings, the necessity of packing the castings in pots is avoided and there should be no objectionable scaling of their surface.

A continuous process of annealing is made use of by a few plants. The installation consists of a tunnel kiln, the length of which may vary from 200 to 350 ft. The stands are loaded on short cars which when in the kiln form its bottom. The cars on passing through the tunnel are subjected to a gradation of temperature so timed as to correspond to the heating cycle required. In this process the annealing time is shortened by about two days.

An electric resistance oven of a semi-continuous type has been designed in which malleablizing can be accomplished in some three to four days. In one type of such an oven, the floor area of the car upon which the castings are placed is of the same rectangular dimensions as the opening in the under part of the oven, and when in position comprises the oven bottom and forms a tight seal which effectively prevents any access of air to the inside of the oven. After the annealing has been completed, the car is removed and replaced by another loaded car which is quickly raised in position. In this manner the oven can be kept quite close to the soaking temperature, saving both time and electric current.

Within the past several years various novel plans for annealing oven lay-outs have been developed, and while construction details are radically different in the individual designs, fuel economy and a shortening of the annealing cycle is attempted in each case through the use of an arrangement of separate chambers so connected by flues that the heat given up during the cooling of the castings from their soaking temperature in one chamber is heating castings in another chamber that is being brought to a soaking temperature.

*Finishing:*

On removal of the stands from the annealing ovens, the castings are separated from the packing material and cleaned in tumbling barrels or by sand blasting. The castings are then sorted and where necessary are ground. Inspection will then discard defectives due to breakage, poor annealing and other causes.

Rather generally, castings are distorted somewhat in heat treatment due in part to the relief of casting strains, but mainly to deformation by their own weight or that of other castings while hot. They are therefore usually straightened after annealing, either by hand, in dies under a drop hammer or in presses, as may be most practical. This straightening is incidentally an insurance against the accidental shipment of unannealed material. Heavy "blocky" castings would be difficult to straighten but fortunately are rather immune to warpage.

## FALLACIOUS IDEAS CONCERNING MALLEABLE IRON

Certain fallacious ideas concerning malleable iron have gained credence through statements that have appeared in the past by writers ignorant of the true character of the product or unfamiliar with the details of the process. Two of the most widespread of these are:

1. That the malleablizing process is not effective in the case of sections in excess of a thickness of approximately ¾ in.
2. That malleable iron is simply a superior character of gray iron.

*Malleablizing of Thick Sections:*

To claim that metal in a section over ¾ in. thick cannot be completely and uniformly graphitized is to infer that after the soaking temperature of about 1600° F. has been maintained for from 48 to 60 hours, such a section will not have been heated uniformly through. As a matter of fact, sections up to at least 4 in. in thickness can be uniformly heated throughout under the conditions of malleablizing, and graphitization will take place throughout the entire section. Metallographic examinations made on cross-sections 4 in. and larger show no difference in structure at any part of the section, except of course at the surface where decarburization usually takes place.

*Malleable Iron is not a Form of Gray Iron:*

The idea that malleable iron is simply a superior character of gray iron probably originated from the fact that in the early days of the industry a large tonnage of cupola malleable iron was made, and as the cupola is the commonly used means of melting gray iron for castings, the association of ideas is probably accountable for the spreading of this statement.

From the well-known fact that the carbon in malleable iron is present as graphite, erroneous conclusions may have been drawn. It is true that graphite is a common constituent of both gray iron and malleable iron, but it is present in very different forms in the two materials. As may be seen by reference to Fig. 3, graphite is present in gray iron in the form of flakes (primary graphite) varying in size and shape and not uniformly distributed; while in malleable iron it exists as equi-axed masses of "temper" carbon more uniformly distributed, as shown in Fig. 2. Again, while pearlite is a usual constituent in gray iron, it should be entirely absent in malleable iron.

If comparisons are to be made, it is manifest that these should be based upon the respective physical properties of the two materials, as the user is concerned primarily with the ability of a metal to stand up in service and not with its structural or chemical composition, except in so far as the latter will influence the former.

## III. PROPERTIES OF MALLEABLE IRON

The various properties of malleable iron considered in this Symposium are listed in the following tabulation. As a convenient summary, and not as a basis of specification, average values for chemical composition, physical constants and certain mechanical and electrical properties are given therein.

The data from which the averages have been determined are presented under the respective subdivisions of this section, which should be consulted for supplementary information of importance. The information on certain mechanical and magnetic properties, and on machineability, microstructure and heat treatment, does not conveniently lend itself to summary in tabular form, and will be found in the text.

Most of the data herein reported have been furnished specifically for purposes of analysis and summary by the Symposium Committee. Many of the tests, especially those for tensile properties, have extended over a considerable period of time and it was not practicable for investigators to submit to the Committee individual results of hundreds, and sometimes thousands, of such tests. Therefore, in general, the only means of indicating the dispersion or spread of the distribution of values has been to report the maximum and minimum values, although it is appreciated that the range between maximum and minimum is an inadequate measure of such distribution. However, in two instances data were available in sufficient detail that standard

## AVERAGE PROPERTIES OF MALLEABLE IRON.

| | |
|---|---|
| CHEMICAL COMPOSITION | Carbon, 1.00 to 2.00; silicon, 0.60 to 1.10; manganese, under 0.30; phosphorus, under 0.20; sulfur, 0.06 to 0.15 per cent. |
| PHYSICAL CONSTANTS: | |
| Specific Gravity | 7.15 to 7.45. |
| Shrinkage Allowance | See text and Fig. 7. |
| Coefficient of Thermal Expansion | Varies with temperature average value 0.000012 per deg. Cent. (0.0000066 per deg. Fahr.). See Fig. 8. |
| Thermal Conductivity | Varies with temperature; see text. |
| Specific Heat | Varies with temperature; average value between 20 and 100° C., 0.122 calories per gram per deg. Cent. |
| MECHANICAL PROPERTIES: | |
| Tensile Strength | 54 000 lb. per sq. in. |
| Yield Point in Tension | 36 000 lb. per sq. in. |
| Elongation in 2 in. | 18 per cent. |
| Reduction of Area | See text. |
| Modulus of Elasticity in Tension | 25 000 000 lb. per sq. in. See Fig. 9. |
| Compressive Strength | No values reported; see text. |
| Transverse Strength | No values reported; see text. |
| Ultimate Shearing Strength | 48 000 lb. per sq. in. |
| Yield Point in Shear | 23 000 lb. per sq. in. |
| Modulus of Elasticity in Shear | 12 500 000 lb. per sq. in. |
| Modulus of Rupture in Torsion | 58 000 lb. per sq. in. |
| Yield Point in Torsion | 24 000 lb. per sq. in. |
| Brinell Hardness Number | 115; range, 100 to 140. |
| Charpy Impact Value | 7.75 ft-lb. |
| Izod Impact Value | See text. |
| Wedge Test for Impact | See text. |
| Fatigue Endurance Limit | See text. |
| Effect of Temperature | See text. |
| ELECTRICAL AND MAGNETIC PROPERTIES: | |
| Resistivity | 28 to 37 microhms per centimeter cube. See Fig. 10. |
| Magnetization Properties | See text and Figs. 11 and 12. |
| Magnetic Hysterisis | See text and Fig. 13. |
| MACHINEABILITY | See text. |
| MICROSTRUCTURE | See text and Figs. 14 to 21. |
| HEAT TREATMENT | See text. |

deviations from average values could be computed, namely, in discussing range of tensile properties in Section IV[1] and in tabulating the tensile properties of higher strength iron in Section V,[2] and in these instances the values presented give much more accurate information on the "frequency distribution" of the available data.

## CHEMICAL COMPOSITION

It will be clear from the description of the manufacture of malleable iron castings that the product consists of a mixture of iron, in the form of ferrite, and free carbon or graphite in the form of "temper" carbon. Combined carbon as cementite or pearlite should not be present in properly annealed malleable iron. It will also be evident that the elements silicon, manganese, phosphorus and sulfur are present. The percentages of these elements and of carbon may vary considerably in different types of iron, depending upon the properties desired, especially tensile strength. (In general, carbon and tensile strength vary inversely.) Also, carbon at the center of a thick section may be nearly the initial total carbon, and very thin sections may be almost completely decarburized. Therefore, the following tabulation of average chemical composition is at best only approximate, and should be interpreted in the light of the preceding statement:

| | | |
|---|---|---|
| Carbon | 1.00 to 2.00 | per cent |
| Silicon | 0.60 to 1.10 | " |
| Manganese | under 0.30 | " |
| Phosphorus | " 0.20 | " |
| Sulfur | 0.06 to 0.15 | " |

Inasmuch as there is no combined carbon in malleable iron, phosphorus may be present in the amount indicated without danger of cold shortness. Also, since malleable iron is not hot-worked, a sulfur content within the range indicated is not objectionable.

## PHYSICAL CONSTANTS

*Specific Gravity:*

The specific gravity of malleable iron varies with the carbon content and is best expressed as ranging from 7.15 to 7.45 at 20° C., referred to water as 1 at 4° C. Values reported by four investigators follow:

SPECIFIC GRAVITY OF MALLEABLE IRON

| | MAXIMUM | MINIMUM | AVERAGE | NUMBER OF TESTS |
|---|---|---|---|---|
| No. 1 | 7.40 | 7.15 | ...... | 37 |
| No. 2 | 7.35132 | 7.33531 | 7.3440 | 5 |
| No. 3 | ....... | ....... | 7.42 | 5 |
| No. 4 | 7.45 | 7.25 | 7.40 | .. |

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[1] See p. 354. [2] See p. 368.

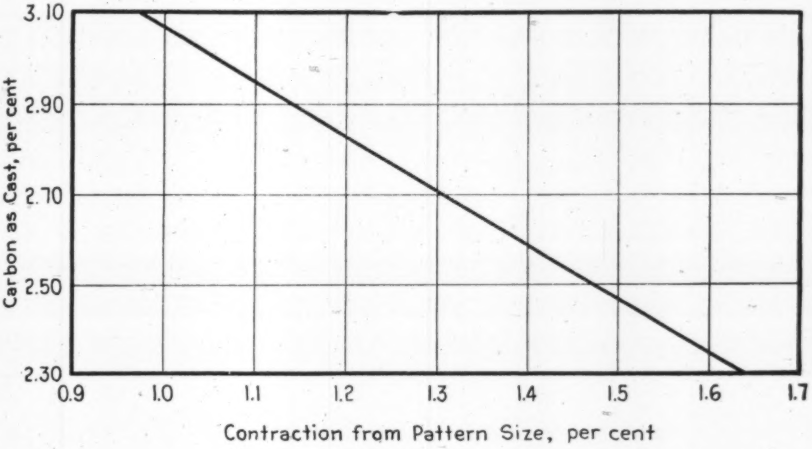

FIG. 7.—Shrinkage Allowance (that is, Net Contraction) from Pattern Size of Malleable Iron Bars.

Based on measurements of about 1000 specimens ½ by ½ by 12 in., annealed under commercial conditions.

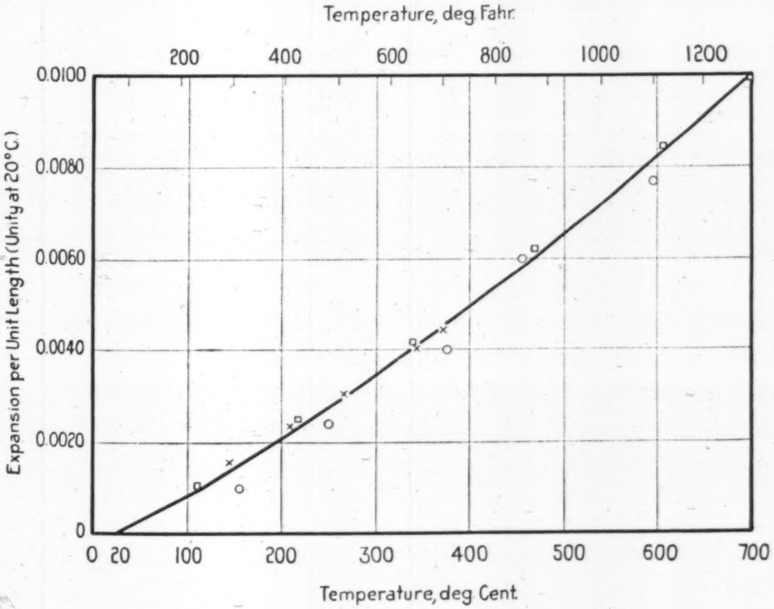

FIG. 8.—Thermal Expansion of Malleable Iron.

*Pattern Makers' "Shrinkage Allowance":*

It has been found by experiment that all white cast iron suitable for malleable castings manufacture shrinks in linear dimension from pattern size by practically the same amount, but since cementite occupies less volume than the iron and carbon formed on its decomposition, the castings expand in annealing to varying degrees dependent upon their carbon content. The variations in specific gravity reported above arise from this cause. The dimensions of a casting from a simple pattern will depend upon the carbon content of the iron as cast and upon the degree of decarburization. Figure 7 was derived from measurements of about 1000 specimens ½ by ½ by 12 in., poured from iron containing 2.40 to 3.10 per cent carbon and annealed under commercial, but not definitely defined, conditions. The curve can, no doubt, be extrapolated to somewhat lower values of carbon.

Consideration must be given, as for all metals, to possible constraining effects which may prevent complicated castings from responding freely to the expected changes of form.

*Thermal Expansion:*

Careful measurements of linear expansion of malleable iron when heated over varying temperature ranges, made by three investigators, have been plotted in Fig. 8, in which the ordinates are expansion per unit length, considering length at 20° C. as unity. Coefficients of expansion for various temperature ranges may be determined from the curve plotted, and will increase slightly with higher temperatures. For temperatures up to approximately 400° C. (750° F.) a value of coefficient of expansion per degree Centigrade sufficiently accurate for ordinary computations is 0.000012 (0.0000066 per deg. Fahr.).

*Thermal Conductivity:*

Attempts have been made to measure the thermal conductivity of malleable iron but the results cannot be said to have been very satisfactory. The following data can be taken only as representing a first approximation at the true values, but may be useful lacking better knowledge:

THERMAL CONDUCTIVITY

| TEMPERATURE, DEG. CENT. | CONDUCTIVITY,[a] $K_t$ |
|---|---|
| 50 | 0.145 |
| 100 | 0.137 |
| 150 | 0.125 |
| 200 | 0.115 |

---

[a] $K_t$ in gram-calories per second per square centimeter per degree Centigrade per centimeter.

*Specific Heat:*

Direct calorimetric determinations of mean specific heat of malleable iron between room temperature (20° C.) and various higher temperatures are reported as follows:

MEAN SPECIFIC HEAT

| TEMPERATURE RANGE, DEG. CENT. | CALORIES PER GRAM PER DEG. CENT. |
|---|---|
| 20 – 100 | 0.122 |
| 20 – 200 | 0.125 |
| 20 – 300 | 0.128 |
| 20 – 400 | 0.133 |
| 20 – 500 | 0.139 |
| 20 – 600 | 0.146 |
| 20 – 700 | 0.159 |

Since malleable iron is a mechanical mixture of approximately pure iron and graphite, its specific heat should be calculable from the data for these substances. Such a calculation gives 0.1102 calories per gram per deg. Cent. at room temperature. This is approximately concordant with the direct observations, but a little lower.

## MECHANICAL PROPERTIES

*Tensile Strength, Yield Point, Elongation:*

Average values of tensile strength, yield point and elongation of malleable iron, determined from the results of upwards of twenty thousand tests reported by seventeen investigators, are as follows:

| | |
|---|---|
| Tensile strength | 54 000 lb. per sq. in. |
| Yield point | 36 000 " " " " |
| Elongation in 2 in | 18 per cent |

Reference should be made to Table I for details, showing the average, maximum and minimum values reported by each investigator. It is desired to point out that these tests cover malleable irons of several more or less well-defined ranges of strength and ductility, and the values reported should be interpreted accordingly. For this reason, it would be more misleading than helpful to attempt to indicate from these data a range of tensile strength, yield point and elongation values that can be expected in ordinary commercial practice.

However, it has been possible to make a statistical analysis of another set of data giving tensile strength, yield point and elongation of 5000 bars from heats made to meet standard specifications. These data are presented in Section IV[1]; average values and standard deviations are reported, together with a brief discussion on the distribution of the observed values. While not directly applicable to the particular data given in Table I, the analysis of range of values for these 5000 tests yields interesting supplementary information, especially in view of the close agreement between the averages for tensile strength, yield point and elongation in the two sets of data.

---

[1] See p. 354.

*Reduction of Area:*

This constant is infrequently determined and values on the usual "as cast" specimens are not especially useful. Machined specimens permitting accurate determinations are not usually prepared. Malleable iron stretches throughout the length of the specimen with little local "necking." On standard specimens ground cylindrical before annealing, values for reduction of area of 18 to 23 per cent may be regarded as representative of material of standard quality.

TABLE I.—DATA ON TENSILE PROPERTIES OF MALLEABLE IRON COMPILED FROM SEVENTEEN SOURCES.

| Investigator | Tensile Strength, lb. per sq. in. | | | Yield Point, lb. per sq. in. | | | Elongation in 2 in., per cent | | | Number of Tests |
|---|---|---|---|---|---|---|---|---|---|---|
| | Maximum | Minimum | Average | Maximum | Minimum | Average | Maximum | Minimum | Average | |
| No. 1... | 59 000 | 45 000 | 54 000 | .... | .... | .... | 26.0 | 10.0 | 18.0 | 6000 |
| No. 2... | 58 500 | 53 000 | 56 250 | 39 600 | 35 000 | 36 700 | 27.5 | 15.0 | 20.5 | 4000 |
| No. 3... | 56 880 | 50 000 | 52 460 | 37 800 | 34 450 | 37 310 | 21.5 | 10.5 | 16.3 | 200 |
| No. 4... | 55 850 | 47 850 | 52 890 | 37 600 | 32 300 | 34 650 | 22.0 | 13.5 | 17.9 | 75 |
| No. 5... | 62 140 | 54 400 | 57 920 | 43 020 | 34 960 | 38 230 | 30.4 | 19.5 | 25.7 | 1210 |
| No. 6... | 62 860 | 52 150 | 56 350 | 39 280 | 35 780 | 37 380 | 28.5 | 15.0 | 21.8 | 300 |
| No. 7... | 56 000 | 50 000 | 53 000 | 37 000 | 35 000 | 36 000 | 27.5 | 10.0 | 20.0 | 50 |
| No. 8... | 58 000 | 50 000 | 55 000 | 40 000 | 30 000 | 35 000 | 25.0 | 10.0 | 20.0 | .... |
| No. 9... | 61 300 | 49 000 | 55 000 | 38 000 | 32 000 | 35 000 | 31.0 | 10.0 | 19.0 | 3200 |
| No. 10... | 59 800 | 50 000 | 53 970 | .... | .... | .... | 28.0 | 10.0 | 17.5 | 289 |
| No. 11... | 60 600 | 46 600 | 52 670 | 40 400 | 36 940 | 38 700 | 21.5 | 8.0 | 14.2 | 36 |
| No. 12... | 58 000 | 50 000 | 53 000 | 38 000 | 30 000 | 35 000 | 25.0 | 10.0 | 12.0 | 500 |
| No. 13... | 62 000 | 51 000 | 53 000 | 39 800 | 35 000 | 36 000 | 32.0 | 15.0 | 18.0 | .... |
| No. 14... | 56 640 | 45 550 | 51 170 | .... | .... | .... | 22.0 | 9.0 | 14.6 | 107 |
| No. 15... | 61 500 | 45 000 | 53 710 | 41 000 | 33 500 | 36 530 | 33.0 | 13.0 | 18.0 | 3300 |
| No. 16... | 58 000 | 50 500 | 55 500 | 38 500 | 34 500 | 36 500 | 28.0 | 10.5 | 22.0 | 540 |
| No. 17... | 56 160 | 50 480 | 52 830 | 37 070 | 35 030 | 35 990 | 23.0 | 15.0 | 17.5 | 59 |
| | Average Tensile Strength........ | | 54 040 | Average Yield Point........... | | 36 350 | Average Elongation | | 18.4 | |

NOTE 1.—All tests in this tabulation are reported on the A.S.T.M. standard test specimen for malleable castings, tested in the "as-cast" condition.

NOTE 2.—The iron is air-furnace iron in all but two cases; the irons reported by No. 1 were made some by the air-furnace process and some by the electric-furnace process; and the irons reported by investigator No. 5 were made by the open-hearth process.

NOTE 3.—The data reported by No. 1 are the average of tests in 10 foundries. The data reported by No. 15 are the average of tests in 17 foundries.

*Modulus of Elasticity in Tension; Stress-Strain Curves:*

The modulus of elasticity of malleable iron in tension is about 25,000,000 lb. per sq. in. The determination of this modulus involves accurate measurement of load and elongation. Measurement of the latter is complicated by residual strains that may be present in the cast test bars, and for reliable determinations carefully straightened specimens should be used. Stress-strain curves obtained by the U. S. Bureau of Standards upon straightened specimens are shown in Fig. 9. Elongations were measured with a Ewing extensometer, which averages the deformations on diametrically opposite sides of the specimen. These curves indicate a value of the modulus of elasticity closely approximating 25,000,000 lb. per sq. in.

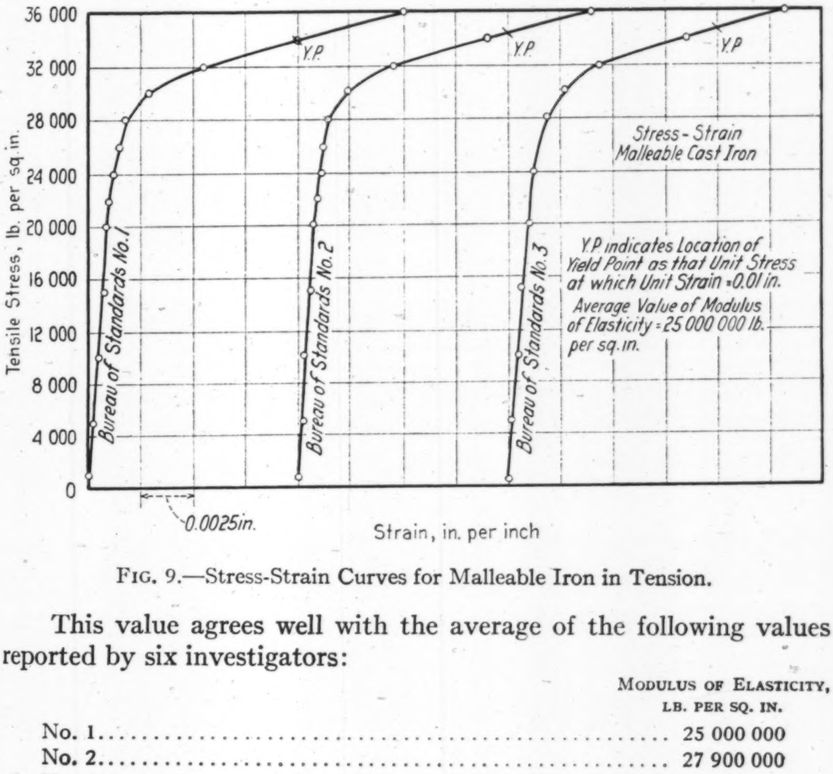

FIG. 9.—Stress-Strain Curves for Malleable Iron in Tension.

This value agrees well with the average of the following values reported by six investigators:

| | MODULUS OF ELASTICITY, LB. PER SQ. IN. |
|---|---|
| No. 1 | 25 000 000 |
| No. 2 | 27 900 000 |
| No. 3 | 24 000 000 |
| No. 4 | 25 000 000 |
| No. 5 | 24 500 000 |
| No. 6 | 24 000 000 |
| Average value | 25 070 000 |

*Compressive Strength; Transverse Strength:*

Because of its ductile character, malleable iron seldom or never actually fails under either compression or transverse loading. In compression the material flows indefinitely, and in cross-bending it deforms to an extent rendering the part useless in service before fracturing.

Careful work on a limited number of specimens has confirmed the plausible supposition that the modulus of elasticity in compression is very near that in tension; for purposes of design from the deflection viewpoint a value of 25,000,000 lb. per sq. in. for this constant may be used. In compression the material takes a permanent set of 1 per cent at about 28,000 lb. per sq. in.

For actual failure in cross-bending, an apparent maximum fiber stress of the order of magnitude of twice the tensile strength is required.

*Shearing Strength:*

In direct shear, malleable iron has an ultimate strength of about 48,000 lb. per sq. in. (approximately nine-tenths of the tensile strength) and a yield point of about 23,000 lb. per sq. in. Values reported by four investigators are tabulated below:

SHEARING STRENGTH OF MALLEABLE IRON.

| | Ultimate Shearing Strength, lb. per sq. in. | | | Yield Point in Shear, lb. per sq. in. | | | Number of Tests |
|---|---|---|---|---|---|---|---|
| | Maximum | Minimum | Average | Maximum | Minimum | Average | |
| No. 1... | ..... | ..... | $\frac{9}{10}$ tens. str.[a] | ..... | ..... | ..... | 25 |
| No. 2... | 51 400 | 47 400 | 49 780 | 25 200 | 19 400 | 23 050 | 10 |
| No. 3... | 48 000 | 40 000 | 45 000 | 25 000 | 20 000 | 23 000 | 5 |
| No. 4... | 52 200 | 50 100 | 51 500 | 24 950 | 23 050 | 23 900 | 5 |
| Average Shearing Strength.......... | | | 48 760 | Average Yield Point........... | | 23 300 | |

---

[a] Not included in average.

The shearing modulus of elasticity ranges from 10,000,000 to 15,000,000 lb. per sq. in.; 12,500,000 is a good average value.

*Torsional Strength:*

Torsion tests of malleable iron are seldom made. From results reported by two investigators, tabulated below, the value of modulus of rupture in torsion may be taken as about 58,000 lb. per sq. in. and the yield point as about 24,000 lb. per sq. in.

TORSIONAL STRENGTH OF MALLEABLE IRON.

| | Modulus of Rupture in Torsion, lb. per sq. in. | | | Yield Point in Torsion, lb. per sq. in. | | | Total Average Twist, deg. | Number of Tests |
|---|---|---|---|---|---|---|---|---|
| | Maximum | Minimum | Average | Maximum | Minimum | Average | | |
| No. 1[a] | ..... | ..... | 58 000 | ..... | ..... | 23 000 | 360[c] | 2 |
| No. 2[b] | 59 940 | 56 380 | 58 040 | 27 040 | 26 250 | 26 760 | 790[d] | 3 |
| Average Modulus of Rupture............. | | | 58 000 | Average Yield Point....... | | 24 850 | | |

---

[a] Test specimen: bar 0.9 in. in diameter, 5-in. gage length. Average tensile strength of same material, 51,000 lb. per sq. in.

[b] Test specimen: bar 1.0 in. in diameter, 10-in. gage length.

[c] Twist over 5-in. gage length.

[d] Twist over 10-in. gage length.

*Brinell Hardness:*

The Brinell hardness of malleable iron is usually about 115 and values may range from about 100 to about 140, as shown in the following tabulation of tests from four investigators:

BRINELL HARDNESS OF MALLEABLE IRON.

| | MAXIMUM | MINIMUM | AVERAGE | NUMBER OF TESTS |
|---|---|---|---|---|
| No. 1 | 143 | 105 | 115 | 25 |
| No. 2 | 130 | 100 | 110 | 25 |
| No. 3 | 143 | 131 | 137 | 10 |
| No. 4 | 130 | 110 | 120 | 5 |

The Brinell hardness is in large measure a function of the free carbon content. There exists a general relation that metal of high tensile strength is usually higher in Brinell hardness and *vice versa.* The correlation is, however, not sufficiently close to use Brinell tests in substitution for tension tests. An increase in tensile strength of 10,000 lb. per sq. in. is accompanied by an increase in hardness of only about 15, while the maxima and minima of hardness values for a given tensile strength often depart by this much from the average.

Nor is Brinell hardness a useful indication of machineability unless the observed hardness falls very far above the range of values given above. The Brinell test will detect an unannealed or enormously under-annealed product, but will not differentiate between two materials of nearly similar and normal machineability. Nor will it detect pearlitic rims of sufficient magnitude to greatly affect machineability.

*Impact Strength:*

Charpy impact values for malleable iron are seldom determined. Only two investigators have reported values, as follows:

CHARPY IMPACT VALUES OF MALLEABLE IRON (FT-LB.).

| | MAXIMUM | MINIMUM | AVERAGE | NUMBER OF TESTS |
|---|---|---|---|---|
| No. 1 | .. | .. | 7.5 | 100 |
| No. 2 | 10 | 7 | 8 | ... |

An average value of 7.75 is the best that can be reported.

One investigator reports Izod impact values on malleable iron, as follows:

IZOD IMPACT VALUES OF MALLEABLE IRON (FT-LB.).

| MAXIMUM | MINIMUM | AVERAGE | NUMBER OF TESTS |
|---|---|---|---|
| 12.9 | 7.9 | 9.3 | 360 |

The so-called "wedge test" is frequently made on malleable iron as a general indication of the ability of the product to withstand impact. Following is a brief description of this test:

A wedge of the material 6 in. long by 1 in. wide, tapering in thickness from $\frac{1}{2}$ in. at the butt to $\frac{1}{16}$ in. at the tip or thin end, is subjected to successive blows of a 21-lb. tup falling from a height of $3\frac{1}{3}$ ft., representing energy of 70 ft-lb. The thin end is first bent over for a length of about $\frac{1}{2}$ in. by means of a hand hammer, starting a "curl." Then the wedge is firmly secured, in approximately a vertical position, at the butt end in a clamp which forms part of an anvil. The tup strikes the curl at the thin end of the wedge, and the clamp is so fitted that the wedge can be gradually tilted from a vertical position in order that the tup may always strike the highest point of the curl. The number of blows required to produce the first sign of fracture is recorded. Obviously the energy of each succeeding blow increases slightly, as the distance through which the tup falls increases by the amount of deformation of the wedge. While the test is distinctly an arbitrary one, it has the merit of being easily and cheaply made, and it seems to be a fairly reliable measure of the shock-resisting qualities of the metal, giving results consistent with the observations of practical experience. It is customary to end the test at 30 blows if no fracture has developed by that time.

Following are the results of wedge tests made by three investigators:

WEDGE TESTS OF MALLEABLE IRON FOR RESISTANCE TO IMPACT

| | NUMBER OF BLOWS AT 70 FT-LB. | | | NUMBER OF TESTS |
|---|---|---|---|---|
| | MAXIMUM | MINIMUM | AVERAGE | |
| No. 1 | 37 | 24 | 30 | 1210 |
| No. 2 | 30 | 20 | 25 | .... |
| No. 3 | 30 | 5 | 26 | 500 |
| Average | | | 27 blows | |

A detail record of the 500 tests by investigator No. 3 gives some interesting data as to distribution of number of blows:

| NUMBER OF BLOWS TO PRODUCE INCIPIENT FRACTURE | NUMBER OF WEDGES | PERCENTAGE OF TOTAL | NUMBER OF BLOWS TO PRODUCE INCIPIENT FRACTURE | NUMBER OF WEDGES | PERCENTAGE OF TOTAL |
|---|---|---|---|---|---|
| 5 | 1 | 0.20 | 13 | 2 | 0.40 |
| 6 | 8 | 1.60 | 14 | 6 | 1.20 |
| 7 | 11 | 2.20 | 15 | 10 | 2.00 |
| 8 | 13 | 2.60 | 16 | 10 | 2.00 |
| 9 | 5 | 1.00 | 17 | 8 | 1.60 |
| 10 | 6 | 1.20 | 18 | 1 | 0.20 |
| 11 | 5 | 1.00 | 30 | 409[a] | 81.80 |
| 12 | 5 | 1.00 | | | |

---

[a] Test discontinued at 30 blows; none of the 409 specimens showed signs of fracture.

*Fatigue Endurance Limit:*

Only one careful determination of fatigue endurance limit of malleable iron is known. This was made on iron of slightly above the minimum properties of A.S.T.M. Specifications A 47 – 30, using the rotating-beam (Farmer type) machine—producing alternate tension and compression in flexure—on a specimen 0.40 in. in diameter, the radius at the point of failure (minimum diameter) being about 3 in. An endurance limit between 25,000 and 26,000 lb. per sq. in. was indicated, thus agreeing with the frequently observed ratio of endurance limit to tensile strength of wrought ferrous metals of approximately 0.5.

*Effect of Temperature on Properties:*

If malleable iron is heated above the lower critical point, about 760° C. (1400° F.), carbon redissolves and the character of the material changes.[1] Hence the use of malleable iron at temperatures over about 1200 to 1300° F. should be avoided.

Malleable iron retains its normal "room temperature" tensile strength at temperatures up to about 800° F.; above that temperature the strength decreases rapidly, being at 1200° F. only one-fourth to one-fifth the tensile strength at room temperature. The elongation is also practically constant up to 800 or 900° F., and rises sharply above the latter temperature.[2]

A few tests on impact strength or malleable iron at temperatures from about $-75°$ to $+390°$ F. have been reported by Schwartz.[3] These tests show impact resistance to be apparently unaffected by change in temperature from room temperature (70° F.) to 390° F., but to drop sharply below 70° F. to Charpy values as low as 3 ft-lb. at $-75°$ F. compared to about 8 ft-lb. at 70° F.

Malleable iron, unless heated above the critical temperature, is immune from growth at high temperatures by graphitization, and practically so from growth by oxidation of silicon or migration of gases. In fact, there appears to be no growth of malleable castings at temperatures for which they are intended to be used. Similarly, within the critical temperature there is little tendency to oxidation at high temperatures.

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[1] See section on Heat Treatment, p. 353.

[2] See Fig. 16, paper by J. W. Bolton and H. Bornstein on "Effect of Elevated Temperatures on Certain Mechanical Properties of Gray Cast Iron and Malleable Iron," Symposium on Effect of Temperature on the Properties of Metals, published jointly by the American Society for Testing Materials and the American Society of Mechanical Engineers (1931).

[3] *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 164 (1924).

## ELECTRICAL AND MAGNETIC PROPERTIES

*Resistivity:*

The resistivity of malleable iron varies with carbon content and ranges from about 28 to 37 microhms[1] per centimeter cube. Values reported by two investigators follow:

| | Resistivity (microhms per Centimeter Cube) | | | |
|---|---|---|---|---|
| | Maximum | Minimum | Average | Number of Tests |
| No. 1 | 37 | 28 | 30.5 | 37 |
| No. 2 | 35.02 | 31.47 | 33.28 | 9[a] |

[a] Determined at 27.5° C. (81.5° F.).

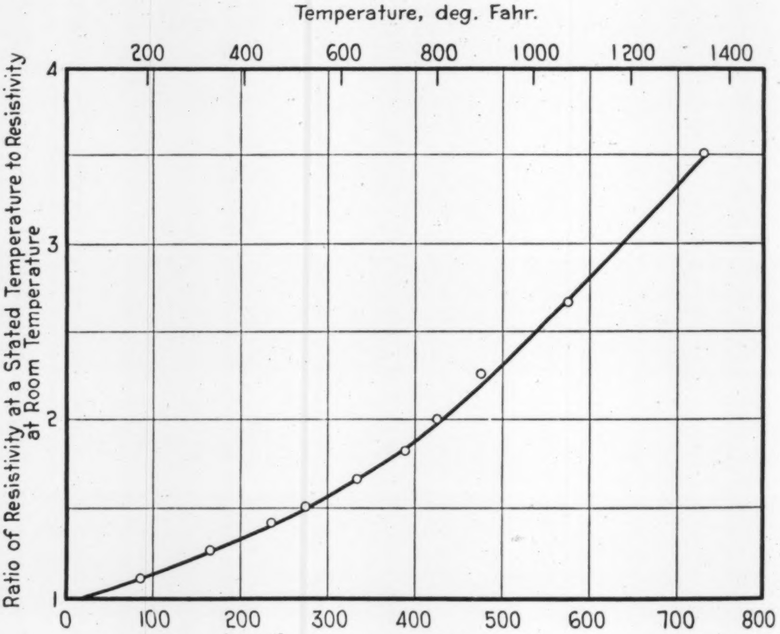

FIG. 10.—Effect of Temperature on Resistivity of Malleable Iron.

The resistivity of malleable iron increases with the temperature as shown in Fig. 10. This curve is based on very careful measurements on a single sample only. Theoretically, the curve will depend upon the chemical composition of the metal, but in practice the conduction of current is so largely through the metal and so little through the graphite that the curve can be applied without serious error to almost any malleable casting.

Above the $A_1$ point, about 760° C. (1400° F.), carbon is redissolved and the resistivity of the metal is permanently increased.

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[1] One microhm = one-millionth of an ohm.

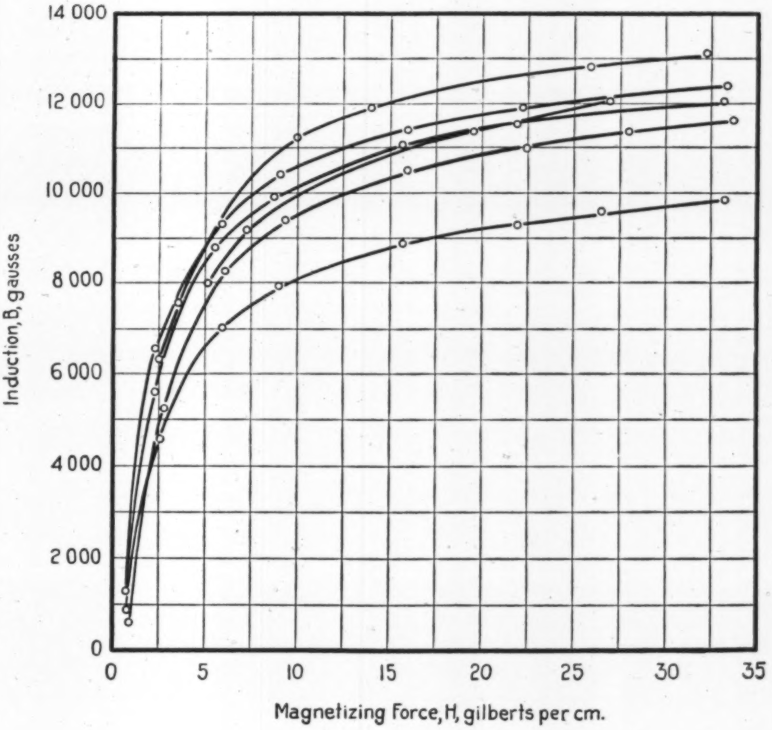
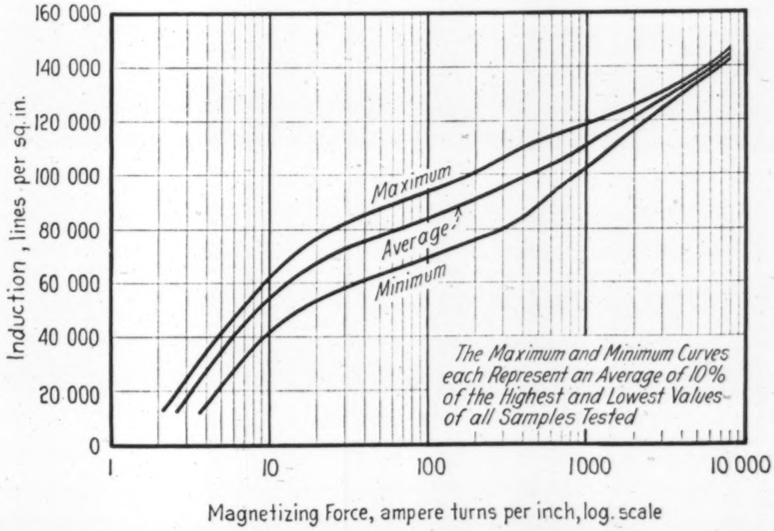


FIG. 11.—Magnetization Curves for Various Malleable Irons.



The Maximum and Minimum Curves each Represent an Average of 10% of the Highest and Lowest Values of all Samples Tested

FIG. 12.—Magnetization Curves for Malleable Iron.

*Magnetization Properties; Permeability:*

Magnetization curves for various malleable irons, as determined by two investigators, are shown in Figs. 11 and 12. In Fig. 11 the magnetizing force, $H$, and the induction, $B$, are recorded in c.g.s. units; in Fig. 12 they are recorded as ampere turns per inch and lines per square inch, respectively. The latter figure covers considerably higher ranges of magnetizing force than the former.

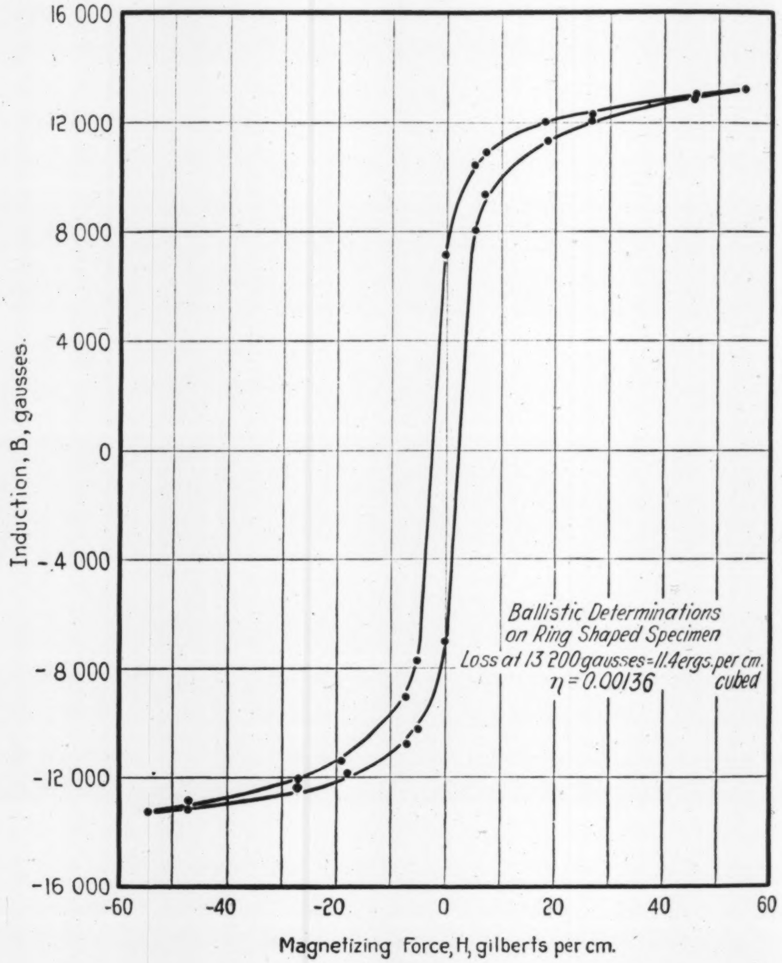


FIG. 13.—Magnetic Hysteresis Curve for Malleable Iron.

The permeability, $\mu$, is expressed by the equation $\mu = B/H$ and may be determined for various inductions from the curves of Figs. 11 and 12.

*Magnetic Hysteresis:*

Figure 13 shows a typical magnetic hysteresis curve for malleable iron.

## MACHINEABILITY

The machineability of malleable castings cannot be dealt with quantitatively since there is no generally accepted test procedure or unit of measurement. It is generally known that as measured by tool life or cutting speed in commercial operations, malleable iron machines more readily than any other ferrous material.

Although there are differences of opinion as to whether variations in tensile properties of malleable iron are accompanied by variations in machineability, it may be said that for practical purposes such variations are not sufficient to contradict the preceding statement for any normal grade of malleable iron. Obviously, metal which accidentally or purposely contains pearlite is not included in this generalization.

Where the machine and tool set-up permits, it is standard practice to take roughing cuts of $\frac{1}{16}$ to $\frac{5}{64}$ in. with a feed of 0.02 to 0.03 in. at cutting speeds of 120 to 135 ft. per minute, and finishing cuts of 0.01 to 0.02 in. with a feed of 0.010 to 0.015 in. at cutting speeds of 200 to 220 ft. per minute. This contemplates the use of ordinary high-speed tools; much higher speeds are obtainable with some of the more modern cutting alloys.

Test data by a particular method of test covering a variety of products is contained in a paper presented before the Society by H. A. Schwartz and W. W. Flagle in 1923.[1] Reference should also be made to the accompanying paper by O. W. Boston on "An Investigation of Methods to Determine the Machineability of Malleable Iron Castings."[2]

## MICROSTRUCTURE

Normally the structure of malleable iron consists of a ground mass of ferrite grains throughout which equi-axed masses of free carbon are distributed (Fig. 14). The term "temper carbon" is applied to this form of carbon present in malleable iron. The carbon may vary somewhat in shape, being sometimes more smoothly rounded than in the illustration and sometimes more irregular.

The surface generally shows the absence of any considerable amounts of free carbon and some vestiges of pearlite can usually be

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[1] H. A. Schwartz and W. W. Flagle, "The Significance of Tool Temperatures as a Function of the Cutting Resistance of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part II, p. 27 (1923).

[2] See p. 388.

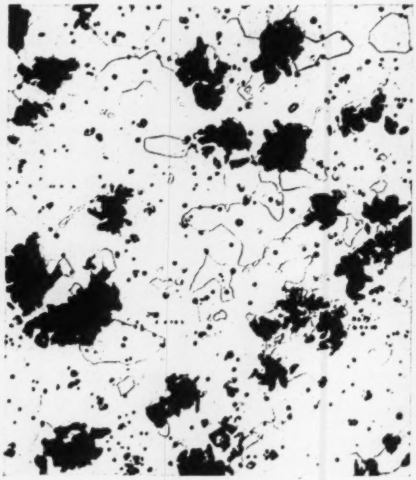

FIG. 14.—Normal Structure.

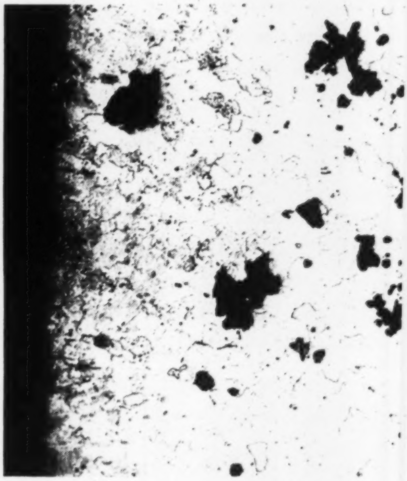

FIG. 15.—Structure at Surface showing Traces of Pearlite.

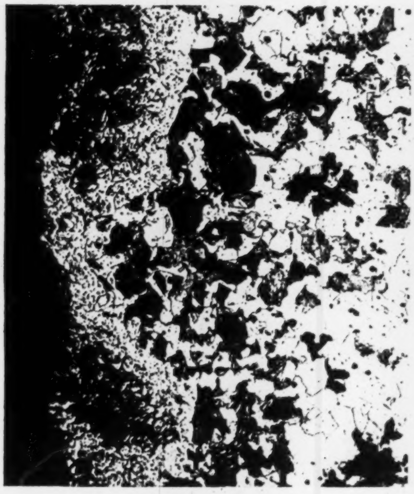

FIG. 16.—Illustrating Incomplete Annealing, with Considerable Pearlite Surviving.

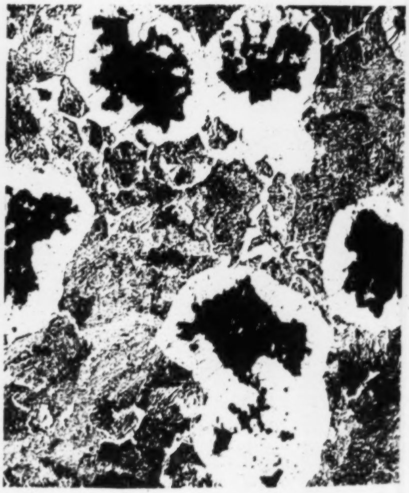

FIG. 17.—Illustrating Effect of Greatly Reduced Annealing Time below $Ar_1$, Showing Pearlite Throughout Mass.

MICROSTRUCTURE OF MALLEABLE IRON (× 100). ETCHED WITH NITAL.

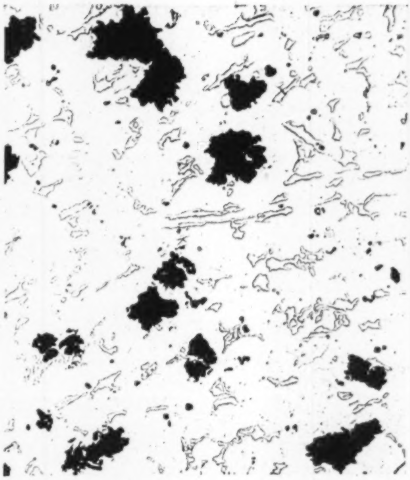

Fig. 18.—Illustrating Effect of Insufficient Annealing above $Ar_1$, Showing Traces of Primary Cementite.

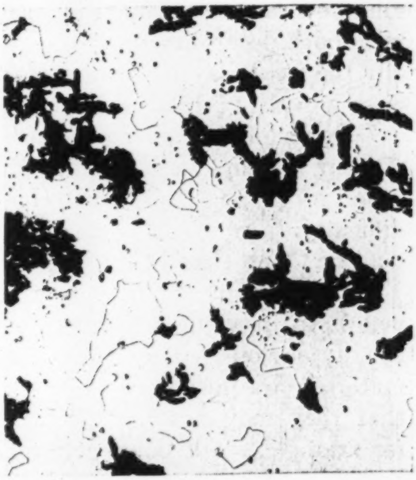

Fig. 19.—Structure Resulting from Presence of Graphite in Original White Iron.

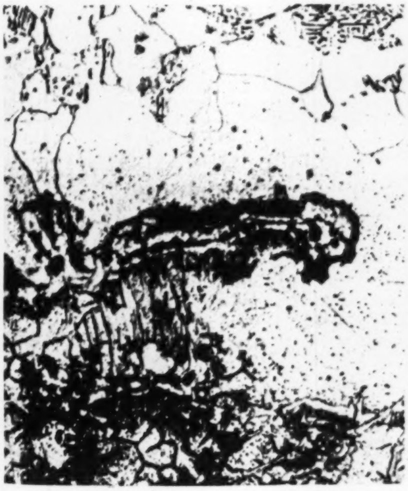

FIG. 20.—Illustrating Oxidized Condition of Malleable Iron (× 500).

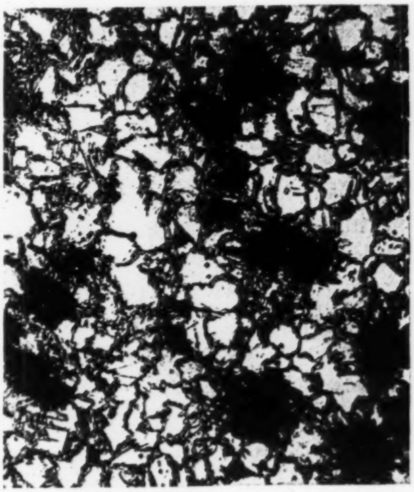

FIG. 21.—Illustrating Recombination of Carbon when Malleable Iron is Reheated Above $Ac_1$.

MICROSTRUCTURE OF MALLEABLE IRON.

All photomicrographs (except Fig. 20) × 100. Etched with Nital.

found on careful search (Fig. 15). It is the usual intention to minimize both decarburization and pearlite as far as possible. Occasionally the ferrite layer is as much as 0.03 to 0.05 in. thick or even more. Such a depth of decarburization may be considered abnormal.

Occasionally much pearlite survives (Fig. 16) representing a significant incompleteness of annealing, sometimes with accompanying "picture frame" and usually with noticeably increased machining difficulty. In defective material very much wider pearlitic bands may be found than in the figure.

Occasionally, with greatly reduced annealing time in the range below $Ar_1$[1] pearlite is found throughout the mass, the graphite being usually surrounded by "bull's eyes" of ferrite (Fig. 17). Certain materials are purposely made of this structure and it is not implied that they may not have valuable engineering properties; rigidity, for example. The point here made is that such material is not malleable iron in the sense of this Symposium and all its properties will fall outside the range here considered.

Iron held insufficiently long above $Ar_1$ will contain vestiges of primary cementite (Fig. 18). Very minute amounts of this phase are probably of little practical importance, but anything beyond traces may be considered evidence of under-annealing.

White iron containing graphite produces very imperfect malleable iron and such primary graphite remains unaltered after annealing. Figure 19 shows in the same field the equi-axed grains of temper carbon normal to malleable iron and the flaky graphite crystals of what was originally a mottle in the hard iron.

When iron becomes oxidized during or before annealing, as in shrinks or sometimes near the surface under strongly oxidizing conditions, cementite becomes stable. Figure 20 shows an area near the surface of a casting showing this defect. The dark areas are oxides and the ferrite grains show a speckled appearance, characteristic of oxidation. The white areas are massive carbides, supposed to contain oxygen, which refuse to graphitize. Near the middle of the picture is shown a group of very coarse lamella-like pearlite but more broadly spaced, characteristic of this condition. The deeply buried pearlite near the bottom of the picture is nearly normal.

Carbon recombines when malleable iron is reheated above $Ac_1$. The recombination begins at the free carbon grains and passes first up the ferrite grain boundaries (Fig. 21), ultimately diffusing out through the mass of the ferrite grain when the entire mass becomes pseudo-martensitic to sorbitic after cooling.

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[1] The $A_1$ point for malleable iron of usual silicon content is near 760° C. (1400° F.).

## HEAT TREATMENT

In this Symposium we are concerned solely with a material substantially completely converted into ferrite and graphite in the so-called "nodular" form (temper carbon). If such material be heated above the lower critical point, say 760° C. (1400° F.), obviously carbon will redissolve. On subsequent cooling at rates comparable to those of the original annealing process this carbon will re-precipitate in the free state. Faster cooling, leaving insufficient time for the graphitizing reaction, will leave the system in the metastable condition and produce in the metallic matrix structures similar to those of high-carbon steels of similar heat treatment: martensite, pseudomartensite, troostite, sorbite, laminated or granular pearlite or spheroidized cementite. Further modifications are possible in that resolution and diffusion of carbon require time and by control of this element free ferrite can be retained along with these structures.

The physical properties of material so heat-treated will be an integration of the effect of free carbon in interrupting the matrix, the amount of combined carbon determined by the time and temperature of reheating, and the metallographic condition of this carbon fixed by subsequent cooling or drawing treatments.

The properties of the resultant metal will evidently be capable of considerable variation. Since the changes are always essentially the substitution of steel for pure iron accompanied by a decrease in graphitic carbon, the resultant change in properties will be toward higher strengths and lowered ductility. These possible changes furnish a useful means of producing special properties for special purposes from malleable castings.

Other products of similar characteristics are produced by suitable modification of the original annealing cycle of a character to retain combined carbon in some desired metallographic form. A treatment of these metals is beyond the scope of the Symposium.

In common with many other ferrous materials, malleable irons when rather rapidly cooled from the "blue heat" range, say 450° C. (850° F.), as in galvanizing, may suffer from intergranular brittleness. If heated to 675° C. (1250° F.) and quenched, the tendency to become so embrittled is for practical purposes completely destroyed. The theory of this process is not completely understood. It is known, however, that the tendency to this embrittlement is related to chemical composition to such a degree that the fault can be avoided by chemical means.[1]

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[1] W. R. Bean, "Deterioration of Malleable in the Hot-dip Galvanizing Process," *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. LXIX, p. 895 (1923).

## IV. SUPPLEMENTARY DATA AND DISCUSSION OF TENSILE PROPERTIES

### RANGE OF TENSILE STRENGTH, YIELD POINT, AND ELONGATION

There is presented in Table II an analysis of 5000 tension tests of malleable iron test bars made by Enrique Touceda for the Malleable Iron Research Institute, the bars having been taken over the period from May to November, 1930, from random heats of 52 companies in the Institute. These heats are representative of the so-called "standard specification" malleable iron, that is, minimum tensile strength of 50,000 lb. per sq. in. and minimum elongation in 2 in. of 10 per cent. The number of bars showing tensile strengths falling within successive 500-lb. ranges from 45,000 to 60,500 lb. per sq. in., and the percentage of the total falling within each 500 lb. range, are shown. Similar information is given for yield point in 1000-lb. ranges from 33,000 to 41,000 lb. per sq. in., and for percentage of elongation in ranges of 2.5 per cent from 10 to 30 per cent.

TABLE II.—ANALYSIS OF 5000 TENSION TESTS OF MALLEABLE IRON BARS.

Tensile Strength

| Range of Values, lb. per sq. in. | Number of Specimens | Percentage of Total |
|---|---|---|
| Under 45 000 | 0 | 0 |
| 45 000 to 45 499 | 0 | 0 |
| 45 500 to 45 999 | 1 | 0.02 |
| 46 000 to 46 499 | 1 | 0.02 |
| 46 500 to 46 999 | 1 | 0.02 |
| 47 000 to 47 499 | 0 | 0 |
| 47 500 to 47 999 | 3 | 0.06 |
| 48 000 to 48 499 | 3 | 0.06 |
| 48 500 to 48 999 | 5 | 0.10 |
| 49 000 to 49 499 | 7 | 0.14 |
| 49 500 to 49 999 | 16 | 0.32 |
| 50 000 to 50 499 | 106 | 2.12 |
| 50 500 to 50 999 | 183 | 3.66 |
| 51 000 to 51 499 | 212 | 4.24 |
| 51 500 to 51 999 | 260 | 5.20 |
| 52 000 to 52 499 | 341 | 6.82 |
| 52 500 to 52 999 | 398 | 7.96 |
| 53 000 to 53 499 | 442 | 8.84 |
| 53 500 to 53 999 | 485 | 9.70 |
| 54 000 to 54 499 | 503 | 10.06 |
| 54 500 to 54 999 | 464 | 9.28 |
| 55 000 to 55 499 | 423 | 8.46 |
| 55 500 to 55 999 | 335 | 6.70 |
| 56 000 to 56 499 | 261 | 5.22 |
| 56 500 to 56 999 | 220 | 4.40 |
| 57 000 to 57 499 | 159 | 3.18 |
| 57 500 to 57 999 | 71 | 1.42 |
| 58 000 to 58 499 | 40 | 0.80 |
| 58 500 to 58 999 | 32 | 0.64 |
| 59 000 to 59 499 | 10 | 0.20 |
| 59 500 to 59 999 | 9 | 0.18 |
| 60 000 to 60 500 | 8 | 0.16 |
| Over 60 500 | 1 | 0.02 |
|  | 5000 | 100.00 |

Yield Point

| Range of Values, lb. per sq. in. | Number of Specimens | Percentage of Total |
|---|---|---|
| Under 33 000 | 0 | 0 |
| 33 000 to 33 999 | 2 | 0.04 |
| 34 000 to 34 999 | 134 | 2.68 |
| 35 000 to 35 999 | 1243 | 24.86 |
| 36 000 to 36 999 | 2100 | 42.00 |
| 37 000 to 37 999 | 1090 | 21.80 |
| 38 000 to 38 999 | 352 | 7.04 |
| 39 000 to 40 000 | 68 | 1.36 |
| Over 40 000 | 11 | 0.22 |
|  | 5000 | 100.00 |

Elongation in 2 in.

| Range of Values, per cent | Number of Specimens | Percentage of Total |
|---|---|---|
| Under 10 | 46 | 0.92 |
| 10.0 to 12.4 | 290 | 5.80 |
| 12.5 to 14.9 | 483 | 9.66 |
| 15.0 to 17.4 | 995 | 19.90 |
| 17.5 to 19.9 | 978 | 19.56 |
| 20.0 to 22.4 | 939 | 18.78 |
| 22.5 to 24.9 | 763 | 15.26 |
| 25.0 to 27.4 | 326 | 6.52 |
| 27.5 to 30.0 | 171 | 3.42 |
| Over 30 | 9 | 0.18 |
|  | 5000 | 100.00 |

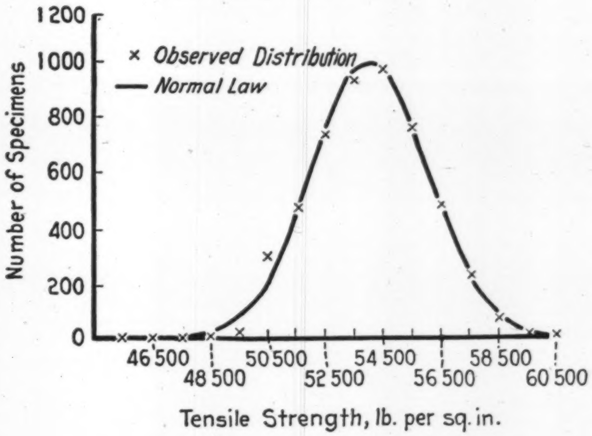

FIG. 22.—Relation for Tensile Strength.

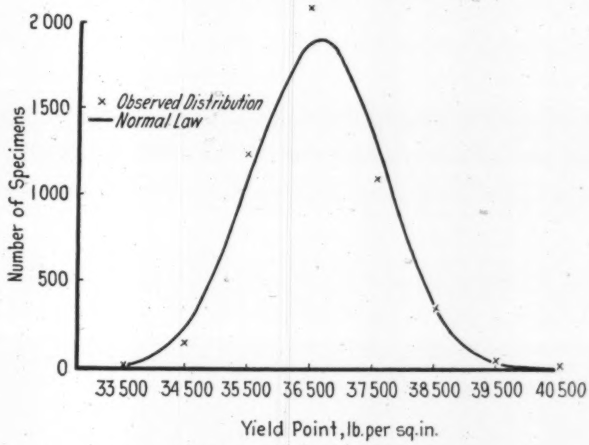

FIG. 23.—Relation for Yield Point.

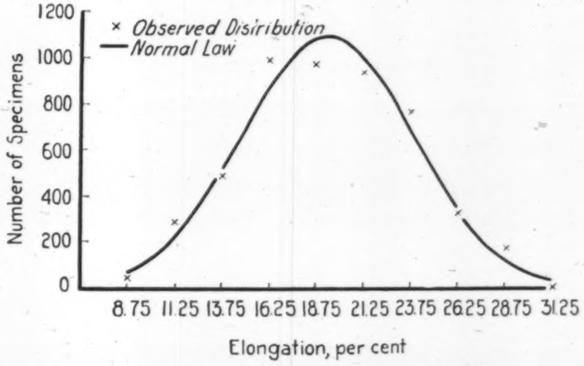

FIG. 24.—Relation for Elongation.

SHOWING RELATION BETWEEN OBSERVED DATA ON 5000 MALLEABLE IRON BARS REPORTED IN TABLE II AND THE NORMAL LAW OF DISTRIBUTION.

The observed averages and standard deviations[1] have been computed[2] from the data in Table II, and are tabulated as follows:

| | TENSILE STRENGTH, LB. PER SQ. IN. | YIELD POINT, LB. PER SQ. IN. | ELONGATION IN 2 IN., PER CENT |
|---|---|---|---|
| Average, ($\overline{X}$) .................... | 54 032 | 36 605 | 19.3 |
| Standard deviation, ($\sigma$) .......... | 1 979 | 1 002 | 4.5 |

| Yield Point Ranges, lb. per sq. in. / Tensile Strength Ranges, lb. per sq. in. | 31 500-31 999 | 32 000-32 499 | 32 500-32 999 | 33 000-33 499 | 33 500-33 999 | 34 000-34 499 | 34 500-34 999 | 35 000-35 499 | 35 500-35 999 | 36 000-36 499 | 36 500-36 999 | 37 000-37 499 | 37 500-37 999 | 38 000-38 499 | 38 500-38 999 | 39 000-39 499 | 39 500-39 999 | 40 000-40 499 | 40 500-40 999 | 41 000-41 499 | 41 500-41 999 | 42 000-42 499 | 42 500 42 999 | Percentage of Total Number within Stated Range of Tensile Strength |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 44 000-44 999 | | | $1^{+}$ | | | | | | | | | | | | | | | | | | | | | 0.2 |
| 45 000-45 999 | | | | | $1^{+}$ | $1^{+}$ | | | | $1^{+}$ | | | | | | | | | | | | | | 0.5 |
| 46 000-46 999 | | | | | | $1^{+}$ | | $1^{+}$ | | | $1^{+}$ | | | | | | | | | | | | | 0.5 |
| 47 000-47 999 | | | | | | | $1^{+}$ | $1^{+}$ | $2^{+}$ | | | | | | | | | | | | | | | 0.6 |
| 48 000-48 999 | | $1^{+}$ | | | | $^{1+}_{1}$ | $1^{+}$ | | 1 | $1^{+}$ | $1^{+}$ | | | | 1 | | | | | | | | | 1.2 |
| 49 000-49 999 | 1 | 1 | | | | $^{1+}_{1}$ | $^{1+}_{3}$ | 2 | $^{1+}_{2}$ | | | | | | | | | | | | | | | 2.0 |
| 50 000-50 999 | | | 5 | 3 | $1^{+}$ | 2 | 12 | 12 | $^{1+}_{12}$ | 9 | 2 | $1^{+}$ | 1 | | | | | | | | | | | 9.3 |
| 51 000-51 999 | | | 2 | 2 | 2 | 2 | 8 | 11 | 15 | $^{1+}_{9}$ | $^{1+}_{7}$ | 2 | 1 | $1^{+}$ | | | | | | | | | | 9.8 |
| 52 000-52 999 | | | 2 | 3 | 3 | 5 | 9 | 15 | 16 | 26 | $^{1+}_{9}$ | $^{2+}_{5}$ | 1 | 1 | 1 | | $1^{+}$ | 1 | | | | | | 15.5 |
| 53 000-53 999 | 1 | | | 2 | 3 | 3 | 5 | 14 | 11 | 11 | 20 | 5 | 7 | 2 | 1 | | | | | | | | | 13.0 |
| 54 000-54 999 | | | | | 1 | 1 | 7 | 2 | 14 | $^{1+}_{14}$ | 17 | 26 | 11 | 9 | 2 | | | 2 | | | 1 | | | 16.5 |
| 55 000-55 999 | | | | | 1 | 2 | 1 | 2 | 8 | 7 | 11 | 16 | 14 | 10 | 4 | 6 | | 2 | | 2 | | | | 13.2 |
| 56 000-56 999 | | | | | | | 1 | | 3 | 4 | 7 | 9 | 11 | 11 | 8 | 6 | 3 | 1 | | | | | | 9.8 |
| 57 000 57 999 | | | | | | | | 1 | | 1 | 4 | 2 | 2 | 2 | 4 | 4 | | | 1 | | | | | 3.2 |
| 58 000-58 999 | | | | | | | | | | | 1 | | | | 3 | 1 | 3 | 2 | 1 | | | | $1^{+}$ | 1.8 |
| 59 000-59 999 | | | | | | | | | | | 1 | | | | 1 | 1 | 1 | | 2 | 3 | 1 | 1 | | 1.7 |
| 60 000-60 999 | | | | | | | | | | | | | | 1 | | | 1 | 1 | | 1 | 2 | | | 0.9 |
| 61 000-61 999 | | | | | | | | | | | | | | | | | 1 | | 1 | | | | | 0.3 |
| Percentage of Total Number within Stated Range of Yield Point | 0.3 | 0.3 | 1.5 | 1.5 | 1.8 | 3.2 | 7.5 | 9.3 | 13.2 | 13.0 | 12.7 | 10.4 | 7.4 | 5.7 | 3.8 | 2.8 | 1.5 | 1.4 | 0.8 | 0.9 | 0.6 | 0.2 | 0.2 | |

FIG. 25.—Relationship Between Tensile Strength and Yield Point of Malleable Iron, from Tests of 653 Bars from 57 Plants.

**Numerals are the number of bars falling within the stated ranges of tensile strength and yield point.**

**Bars marked + showed less than 10 per cent elongation.**

When the average, $\overline{X}$, and standard deviation, $\sigma$, are known, it follows (from Tchebycheff's theorem) that more than 100 $(1 - \frac{1}{t^2})$

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[1] "Standard deviation" is the square root of the mean squares of the deviations of the individual observations from their average value. It is the best measure of variability or dispersion. From the viewpoint of presentation of data, it indicates how the individual values are distributed about the average value. From the viewpoint of estimation, it furnishes a basis for estimating the way in which the observed values in a future sample will be distributed about some fixed value.

[2] The Committee is indebted to W. A. Shewhart, Bell Telephone Laboratories, Inc., for the statistical analysis of these data, the preparation of Figs. 22, 23 and 24, and the discussion thereof.

per cent of the observed values lie within the closed interval $\overline{X} \pm t\sigma$, where $t$ is greater than 1. For example, eight-ninths of the observed values of tensile strength lie within the interval of 54,032 ± 5937 lb. per sq. in.; eight-ninths of the observed values of yield point lie within the interval of 36,605 ± 3006 lb. per sq. in.; and eight-ninths of the observed values of elongation lie within the interval of 19.3 ± 13.5 per cent.

| Yield Point Ranges, lb. per sq. in.<br>Elongation in 2 in., per cent | 31 500-31 999 | 32 000-32 499 | 32 500-32 999 | 33 000-33 499 | 33 500-33 999 | 34 000-34 499 | 34 500-34 999 | 35 000-35 499 | 35 500-35 999 | 36 000-36 499 | 36 500-36 999 | 37 000-37 499 | 37 500-37 999 | 38 000-38 499 | 38 500-38 999 | 39 000-39 499 | 39 500-39 999 | 40 000-40 499 | 40 500-40 999 | 41 000 41 499 | 41 500-41 999 | 42 000-42 499 | 42 500-42 999 | Percentage of Total Number having Stated Elongation |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 4 | | | | | | | | | | | | | | | | | | | | | | | 1 | 0.1 |
| 5 | | | | | | | | | | | | | | | | | | | | | | | | 0.0 |
| 6 | | | | | | | | | | | | | | | | | | | | | | | | 0.0 |
| 7 | | | $1^{+}$ | | | $3^{+}$ | 1 | $1^{+}$ | | $1^{+}$ | $2^{+}$ | $2^{+}$ | | 1 | | | | | | | | | | 1.9 |
| 8 | | | | | $1^{+}$ | | | | $2^{+}$ | 1 | 1 | | $1^{+}$ | | | | | | | | | | | 0.9 |
| 9 | | | | | | $1^{+}$ | $2^{+}$ | $1^{+}$ | $^{1+}1$ | $^{1+}1$ | 1 | 2 | | | | | 1 | | | | | | | 1.9 |
| 10 | | $1^{+}$ | | 1 | | $^{1+}1$ | $^{1+}1$ | $^{2+}5$ | $^{1+}3$ | 7 | 2 | 3 | 4 | 2 | $1^{+}$ | | 1 | | 1 | | | | | 5.8 |
| 11 | | | | | | | 1 | 2 | 6 | 3 | 3 | 4 | 4 | 2 | | | | | | | | | | 3.8 |
| 12 | | 1 | | 2 | | | 4 | 3 | 5 | 5 | 5 | 6 | 2 | | 1 | 1 | 1 | | | | | | | 5.5 |
| 13 | | | | | | | | 2 | $^{1+}3$ | 1 | 1 | | 2 | 1 | | | | | | | | | | 1.7 |
| 14 | | | 1 | | 1 | | $^{1+}2$ | 4 | $^{1+}6$ | 6 | 9 | 3 | 3 | 3 | 2 | 1 | | 1 | | | | | | 6.7 |
| 15 | $1^{+}$ | | 2 | 1 | | $1^{+}$ | 4 | 7 | 9 | 7 | 10 | 10 | 4 | 5 | 6 | 2 | 2 | | | | | | | 10.9 |
| 16 | | | | 1 | 1 | 1 | 1 | 2 | 2 | | 2 | 2 | 2 | 1 | 3 | | | | | | | | | 2.7 |
| 17 | | | 1 | 2 | 1 | 1 | 8 | 9 | 10 | 18 | 11 | 14 | 6 | 5 | 1 | 4 | | 1 | | 1 | 1 | | | 14.4 |
| 18 | | | | | 1 | 2 | 3 | 3 | 4 | 6 | 4 | 3 | 5 | 1 | 1 | 1 | 1 | 2 | | 1 | | | | 5.8 |
| 19 | | | 2 | 1 | 1 | 3 | 7 | 2 | 4 | 6 | 6 | 6 | 2 | 5 | 1 | 1 | 1 | | | | | | | 7.4 |
| 20 | 1 | | 2 | | | 2 | 3 | 5 | 6 | 6 | 7 | | 3 | 4 | 1 | 2 | 1 | | | | | | | 6.6 |
| 21 | | | | | 1 | 1 | 3 | 3 | 4 | 5 | 2 | 4 | 2 | 1 | 1 | | | | | | 1 | | | 4.3 |
| 22 | | | | 1 | 2 | 1 | 2 | 4 | 4 | 3 | 11 | 3 | 3 | 3 | 2 | 3 | | 1 | | 1 | | | | 6.7 |
| 23 | | | | 1 | 1 | 1 | | 3 | 3 | 2 | 1 | 2 | | 1 | 2 | 1 | | | | | | 1 | | 2.9 |
| 24 | | | | | 2 | 1 | | 1 | 4 | 1 | 1 | 1 | 3 | 1 | 1 | 1 | 1 | 1 | 1 | | | | | 3.1 |
| 25 | | | 1 | | | 1 | 4 | 2 | 4 | 1 | 1 | 2 | 1 | | 2 | 1 | 1 | 1 | 3 | 1 | 1 | | | 4.1 |
| 26 | | | | | | | 1 | | | | 2 | 1 | | | | | | 1 | | 1 | | | | 0.9 |
| 27 | | | | | | | | | 2 | 3 | | | | 1 | | | | 1 | | 1 | 1 | | | 1.4 |
| 28 | | | | | | | | | | | 1 | | | | | | | | | | | | | 0.2 |
| 29 | | | | | | | | | | 1 | | | 1 | | | | | | | | | | | 0.3 |
| Percentage of Total Number within Stated Range of Yield Point | 0.3 | 0.3 | 1.5 | 1.5 | 1.8 | 3.2 | 7.5 | 9.3 | 13.2 | 13.0 | 12.7 | 10.4 | 7.4 | 5.7 | 3.8 | 2.8 | 1.5 | 1.4 | 0.8 | 0.9 | 0.6 | 0.2 | 0.2 | |

FIG. 26.—Relationship Between Yield Point and Elongation in 2 in. of Malleable Iron, from Tests of 653 Bars from 57 Plants.

**Numerals are the number of bars falling within the stated ranges of yield point and elongation. Bars marked + showed less than 50,000 lb. per sq. in. tensile strength.**

While it is theoretically possible to find a function that will fit each of these observed distributions accurately, to do so has very little value because it gives no more information than is contained in Table II. It is of interest, however, to see how closely the normal law fits the observed data. This information is indicated schematically in Figs. 22, 23 and 24. While it may be shown analytically that the degree of fit between the observed and theoretical distributions does not warrant the assumption that the observed data came from a "controlled normal universe," these figures illustrate that the integral of the normal law between given limits not extending beyond three times the standard deviation on either side of the average is a fair approximation of the number of observed values lying within this range, for each of the three properties, tensile strength, yield point, and elongation.

Since the bars reported in Table II were being tested for certification to conformity with A.S.T.M. Specifications for Malleable Castings A 47, or their equivalent, it is significant that less than 1 per cent of these 5000 bars showed tensile strengths under 50,000 lb. per sq. in., none showed yield points under 32,500 lb. per sq. in., and less than 1 per cent showed elongations of under 10 per cent; these being the specified minimum limits for these three properties.

There are also presented in Figs. 25 and 26 similar data, likewise furnished by Touceda, from tension tests of 653 malleable iron bars (not included in the 5000 bars reported above) from 57 different malleable plants. In Fig. 25 is shown the relationship between tensile strength and yield point, by noting the number of bars falling within 1000-lb. ranges of tensile strength and 500-lb. ranges of yield point. The percentages of the total number of bars falling within the stated ranges are shown at the right and bottom of the figure, respectively. Bars having less than 10 per cent elongation are indicated. Similarly, in Fig. 26 is shown the relationship between yield point and percentage of elongation in 2 in., using ranges of 500 lb. for yield point and 1 per cent for elongation. The percentages of bars falling within stated ranges of yield points and elongations are shown at the bottom and the right, respectively, and bars having less than 50,000 lb. per sq. in. tensile strength are identified.

## RELATION OF TENSILE STRENGTH TO ELONGATION[1]

The use of a single specification for malleable iron easily suggests the misconception that it is impossible to vary the physical properties of that material. This is by no means true and this metal is now made, as it has been for many years, of varying physical properties to suit different uses or the desires of various consumers.

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[1] Contributed by H. A. Schwartz.

It may therefore be useful to invite attention to a very unusual relation existing between the tensile strength and elongation of malleable iron which is perhaps unique among the commonly used ferrous materials. For most materials the relation holds that the tensile strength and elongation vary inversely, while for malleable iron the stronger product has the higher ductility. This leads to the very interesting conclusions that both factors of energy of rupture, stress and strain, work together and the energy of rupture is not limited by a constantly falling elongation as the tensile strength is raised. As illustrative of this interesting and valuable relationship the following is offered.

The present conclusions are based on certain operating data collected by eleven producers, then members of the American Malleable Castings Association, as a part of the technical activities of that organization.[1] The data are applicable solely to material containing no substantial amounts of combined carbon.

Any malleable plant produces, intentionally and otherwise, variations in chemical composition which are reflected in the resulting tensile properties. The participating companies early in 1927 subdivided their previous six month's or year's production into groups of approximately constant chemical composition and averaged the tensile strength and elongation of each group. The number of specimens reported by each plant ranged from 40 for a small plant, to 1300 for a large plant, the total number reported by all being estimated as perhaps six thousand from eleven plants.

In Fig. 27 the mean tensile properties are plotted for the grand average for all plants by groups. The abscissas are tensile strengths, the ordinates are the corresponding percentages of elongations. All plants were weighted equally in the compilation, although the weighting in proportion to number of heats would not have materially altered the results.

Furthermore, each plant studied the properties of specimens from a single group of nearly constant composition, the group being that which, considering all the plants, represented the greatest number of specimens. Figure 28 shows the relation of tensile strength to percentage of elongation based on this study, the average percentage of elongation for bars of nearly constant tensile strength from heats of this nearly constant composition having been averaged for each plant and the resulting averages combined into a grand average for each strength group, giving equal weight to each plant.

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[1] Permission for its use here has been given by the Association's successor, the Malleable Iron Research Institute, to whom and to whose Board of Directors thanks are due. It is deemed inexpedient to name separately the individual cooperating companies, but thanks are due them even though they remain anonymous.

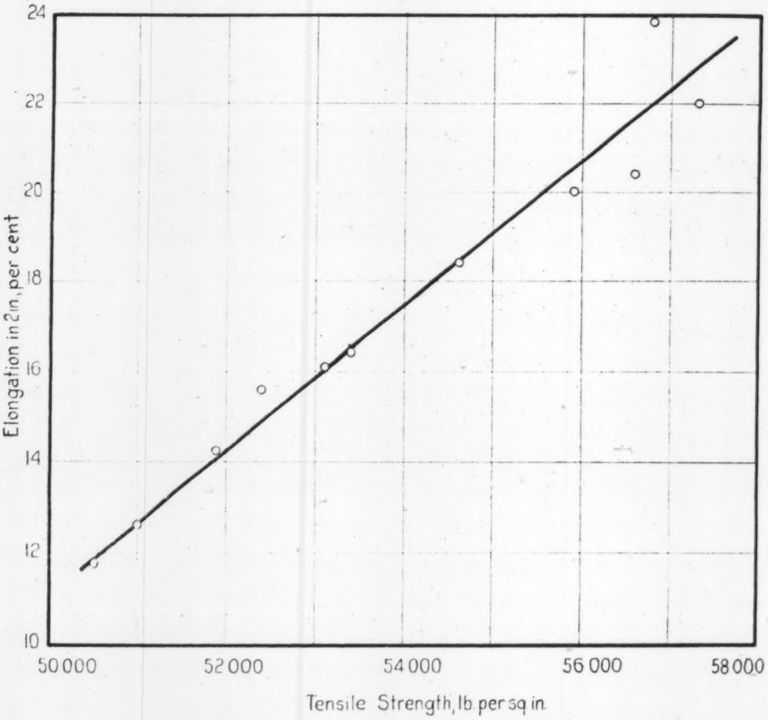

FIG. 27.—Relation of Tensile Strength and Elongation of Malleable Irons of Variable Composition.

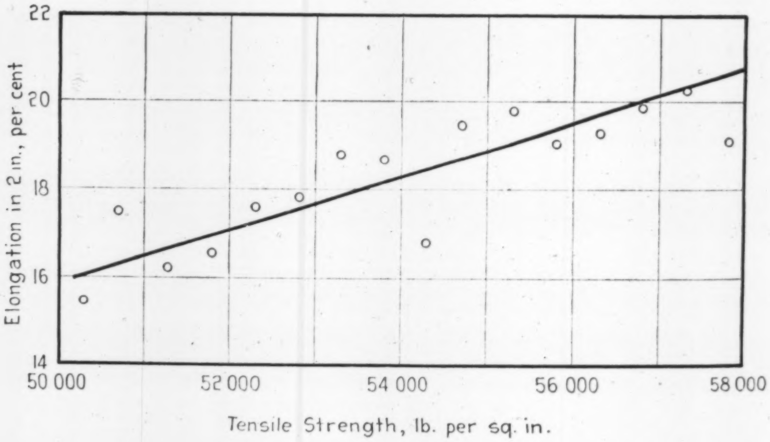

FIG. 28.—Relation of Tensile Strength and Elongation of Malleable Irons of Constant Composition.

The straight trend lines in Figs. 27 and 28 have been drawn in by inspection and not by calculation by the method of least squares. They will, however, suffice in precision for the present purpose. It will be seen that in so far as tensile strength is determined by original composition, an increase of 5000 lb. per sq. in. in tensile strength is accompanied by an increase of about 8 per cent in elongation (Fig. 27). If the variation in tensile strength is due to factors other than the original composition, an increase of 5000 lb. per sq. in. in tensile strength is accompanied by an increase of only about 3 per cent in elongation.

No definite reason for this difference can be assigned. The physical properties of malleable iron are determined mainly by carbon content and this in turn by both original content and decarburization. It is suggested that decarburization affects tensile strength more than it does elongation, possibly through the intervention of oxygen or small amounts of combined carbon. An experimental investigation of the validity of this suggestion is beyond the scope of this compilation.

## EFFECT OF MACHINING AND CROSS-SECTIONAL AREA ON STRENGTH[1]

There is an idea, still held by some, that the strength of malleable iron lies mostly in the decarburized surface metal or "skin" of the casting. While it may be stated generally that decarburization has a favorable influence upon strength and ductility, many test data are available showing that removal of the surface metal does not seriously affect the strength and elongation of malleable iron test bars. The following are selected as typical.

In Table III are presented values of tensile strength, yield point, elongation and reduction of area determined on six sets of malleable iron bars. Each set contained four bars; one bar was cast to $\frac{5}{8}$ in. diameter — the diameter of the standard tension test specimen; the other three bars were cast to diameters of $\frac{3}{4}$, $\frac{7}{8}$ and 1 in., respectively, and machined to $\frac{5}{8}$ in. diameter before testing — that is, with machining cuts $\frac{1}{16}$, $\frac{1}{8}$ and $\frac{3}{16}$ in. deep, respectively.

It will be seen from this table that there is very little difference in either tensile strength or yield point between the "as cast" and machined bars in any of the six sets, and that in four of the six sets there is not much difference in either elongation or reduction of area. In the case of the 1-in. bars, with practically 60 per cent of the section removed, the values reported represent simply the "core" of the metal, yet there is comparatively small difference in yield point and tensile strength compared with the "as cast" bars.

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[1] Prepared from data contributed by H. A. Schwartz and Enrique Touceda.

In another series of tests, bars in sets of six from the same heat were secured from eight different plants, each set therefore being from a different heat. Three bars from each set were tested as cast, while the skin was machined off the other three in excess of usual tolerances for castings that are to be machined. The range in tensile strength of the irons was from an average of 51,200 lb. per sq. in. for all bars from the heat of lowest strength to 58,700 lb. per sq. in. for all bars from the heat of highest strength.

TABLE III.—TESTS OF MALLEABLE IRON TEST BARS IN "AS CAST" CONDITION AND MACHINED.

Iron made by Frazer and Jones Co. in 1920; tests made by Robert W. Hunt Co.

| Set | Diameter of Bar as Cast, in. | How Tested | Tensile Strength, lb. per sq. in. | Yield Point, lb. per sq. in. | Elongation in 2 in., per cent | Reduction of Area, per cent |
|---|---|---|---|---|---|---|
| No. 1 | $\frac{5}{8}$ | As cast | 55 720 | 35 410 | 29.0 | 30.8 |
| | $\frac{3}{4}$ | Machined to $\frac{5}{8}$ in. | 55 380 | 35 350 | 22.5 | 22.6 |
| | $\frac{7}{8}$ | Machined to $\frac{5}{8}$ in. | 54 980 | 34 950 | 21.5 | 21.7 |
| | 1 | Machined to $\frac{5}{8}$ in. | 55 140 | 35 800 | 19.5 | 16.9 |
| No. 2 | $\frac{5}{8}$ | As cast | 55 880 | 35 040 | 25.0 | 26.2 |
| | $\frac{3}{4}$ | Machined to $\frac{5}{8}$ in. | 54 840 | 35 310 | 20.5 | 19.7 |
| | $\frac{7}{8}$ | Machined to $\frac{5}{8}$ in. | 53 740 | 34 980 | 15.5 | 14.2 |
| | 1 | Machined to $\frac{5}{8}$ in. | 53 600 | 34 500 | 14.5 | 13.0 |
| No. 3 | $\frac{5}{8}$ | As cast | 56 580 | 34 600 | 26.0 | 30.8 |
| | $\frac{3}{4}$ | Machined to $\frac{5}{8}$ in. | 55 580 | 35 760 | 24.0 | 24.9 |
| | $\frac{7}{8}$ | Machined to $\frac{5}{8}$ in. | 55 460 | 36 320 | 24.0 | 20 5 |
| | 1 | Machined to $\frac{5}{8}$ in. | 54 820 | 35 370 | 24.0 | 22.1 |
| No. 4 | $\frac{5}{8}$ | As cast | 55 440 | 35 220 | 26.0 | 29.4 |
| | $\frac{3}{4}$ | Machined to $\frac{5}{8}$ in. | 55 260 | 35 960 | 23.0 | 24.3 |
| | $\frac{7}{8}$ | Machined to $\frac{5}{8}$ in. | 54 420 | 35 310 | 20.0 | 21.1 |
| | 1 | Machined to $\frac{5}{8}$ in. | 54 540 | 35 140 | 23.0 | 22 6 |
| No. 5 | $\frac{5}{8}$ | As cast | 55 200 | 35 310 | 26.5 | 20.8 |
| | $\frac{3}{4}$ | Machined to $\frac{5}{8}$ in. | 55 380 | 35 340 | 22.5 | 23.5 |
| | $\frac{7}{8}$ | Machined to $\frac{5}{8}$ in. | 55 020 | 35 340 | 21.5 | 19.2 |
| | 1 | Machined to $\frac{5}{8}$ in. | 53 840 | 35 370 | 24.5 | 22.9 |
| No. 6 | $\frac{5}{8}$ | As cast | 55 120 | 33 720 | 26 0 | 29.2 |
| | $\frac{3}{4}$ | Machined to $\frac{5}{8}$ in. | 54 840 | 35 120 | 26.0 | 25.1 |
| | $\frac{7}{8}$ | Machined to $\frac{5}{8}$ in. | 55 240 | 34 310 | 23.0 | 23.2 |
| | 1 | Machined to $\frac{5}{8}$ in. | 54 920 | 34 650 | 24.0 | 22.6 |

A comparison of the average tensile strengths of the "as cast" with the machined bars of each heat showed higher tensile strength for the machined bars in six heats out of eight, the differences ranging from 35 to 1800 lb. per sq. in. In the two heats in which the "as cast" bars tested higher, the differences were 60 and 460 lb. per sq. in. The "as cast" bars showed slightly higher yield point than the machined bars in four heats and slightly lower in the other four.

The percentage of elongation of the machined bars was higher in seven heats. Reduction of area of the machined bars was higher in six heats.

When specimens of malleable iron of various cross-sectional areas, including both machined and unmachined specimens, are tested, a number of variable factors may influence the physical properties: such as,

1. Decarburization may strengthen the surface metal.
2. The surface metal may be stronger than that at the center, due to the effect on the grain structure of the rate of cooling in freezing.
3. Specimens of small cross-sectional area as cast may be relatively stronger from the same cause.
4. Slower cooling in the larger cross-sections may precipitate primary graphite with disastrous results.
5. The larger cross-sections may not be mechanically sound due to shrinkage. This weakens them as a whole and also weakens the center as compared with the surface.
6. The machined specimen, owing to its uniformity of cross-section, will yield apparently better results, particularly as to elongation and reduction of area, than a rough one.

In 1920, H. A. Schwartz reported[1] the results of experiments to test the effect of several variables. These results are abstracted in the following paragraphs; for more complete discussion the original paper should be consulted.

A set of specimens cast to the form and dimensions shown in Fig. 29 was decided on and after much experimentation that system of gating and feeding was adopted which seemed most certain to exclude variations from the fifth cause referred to above.[2] From this pattern equipment, a number of specimens of each size were cast from metal of the following composition:

| | | |
|---|---|---|
| Carbon | 2.44 | per cent |
| Silicon | 0.76 | " |
| Manganese | 0.180 | " |
| Phosphorus | 0.166 | " |
| Sulfur | 0.099 | " |

Examination of the largest specimens disclosed the absence of primary graphite, thus eliminating variable No. 4. The carbon content of the metal removed by machining was from 0.09 to 0.15 per cent. All the specimens were unusually free from surface defects.

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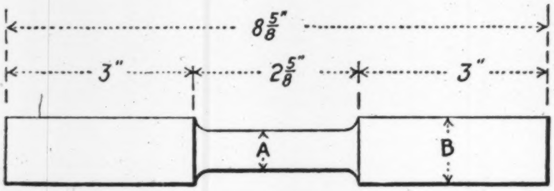
[1] H. A. Schwartz, "Effect of Machining and of Cross-Section on the Tensile Properties of Malleable Cast Iron," *Proceedings*, Am. Soc. Testing Mats., Vol. XX, Part II, p. 70 (1920).

[2] It will incidentally be noted that the test specimens are very similar in general design to the more recently adopted standard specimen for malleable iron now in use (A.S.T.M. Specifications A 47 – 30). The gating and feeding were also very similar to that now commonly used for the standard specimen.

The best specimen of each size was set aside for use without machining, thus minimizing the consequence of variable No. 6 as far as possible, though of course it can not be entirely overcome. One specimen of each size was ground for its entire gage length to a diameter $\frac{1}{8}$ in. less than its nominal diameter as cast. This involved the removal of a little over $\frac{1}{16}$ in. of the surface metal in most cases. The set of specimens originally reserved, the ground set and an additional rough set were then annealed under conditions which assured the greatest possible uniformity throughout the series. After annealing, the ground specimens had, of course, increased about 1 per cent in diameter. The third or rough set of annealed specimens was then machined to the same diameters as the corresponding members of the series that had been ground to size.



| A | B |
|---|---|
| $\frac{1}{4}''$ | $\frac{5}{8}''$ |
| $\frac{3}{8}''$ | $\frac{3}{4}''$ |
| $\frac{1}{2}''$ | $\frac{7}{8}''$ |
| $\frac{5}{8}''$ | 1" |
| $\frac{3}{4}''$ | $1\frac{1}{8}''$ |
| $\frac{7}{8}''$ | $1\frac{1}{4}''$ |
| 1" | $1\frac{3}{8}''$ |
| $1\frac{1}{8}''$ | $1\frac{1}{2}''$ |
| $1\frac{1}{4}''$ | $1\frac{5}{8}''$ |
| $1\frac{3}{8}''$ | $1\frac{3}{4}''$ |
| $1\frac{1}{2}''$ | $1\frac{7}{8}''$ |

FIG. 29.—Dimensions of Test Specimens as Cast.

The two finished series were thus exactly similar to each other except as to the effect of the first variable. The series ground to size before annealing was exactly similar to the unmachined series, except as to the effect of the second and third variables, which are related, and some residue of the effect of the sixth variable which could not be completely overcome. Each series within itself was alike, except with respect to the third variable.

Some of the data from the three series are presented in Figs. 30, 31 and 32. An intercomparison of these data permits of conclusions as to the relative effect of the first three variables. Thus, to study the effect on tensile strength of machining $\frac{1}{16}$ in. off the surface, compare the tensile strength shown in Fig. 32 with the strength of specimens $\frac{1}{8}$ in. larger in diameter shown in Fig. 31. The effect of the single variable of diameter of specimens as cast is shown directly in

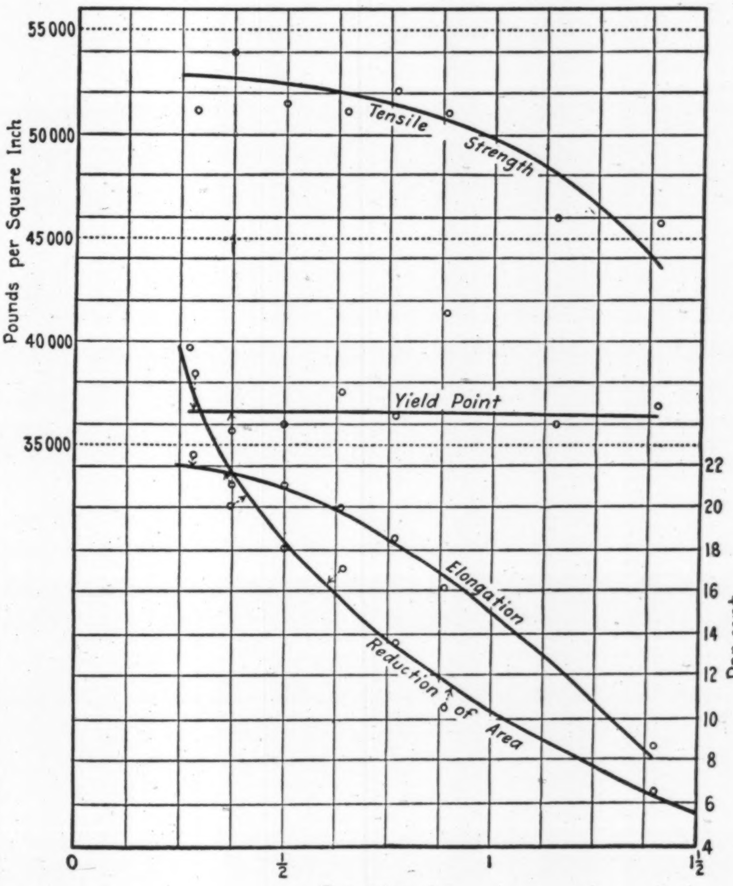


FIG. 30.—Results of Tests on Specimens Ground Before Annealing.

Fig. 32. The combined effect of the three variables on tensile strength amounts to about 7000 lb. per sq. in. for sections $\frac{1}{4}$ in. in diameter or less, and becomes negligible at diameters of $1\frac{1}{4}$ in. or over. The combined effect on elongation is about 3 per cent for small specimens and negligible for diameters above $\frac{3}{4}$ in.

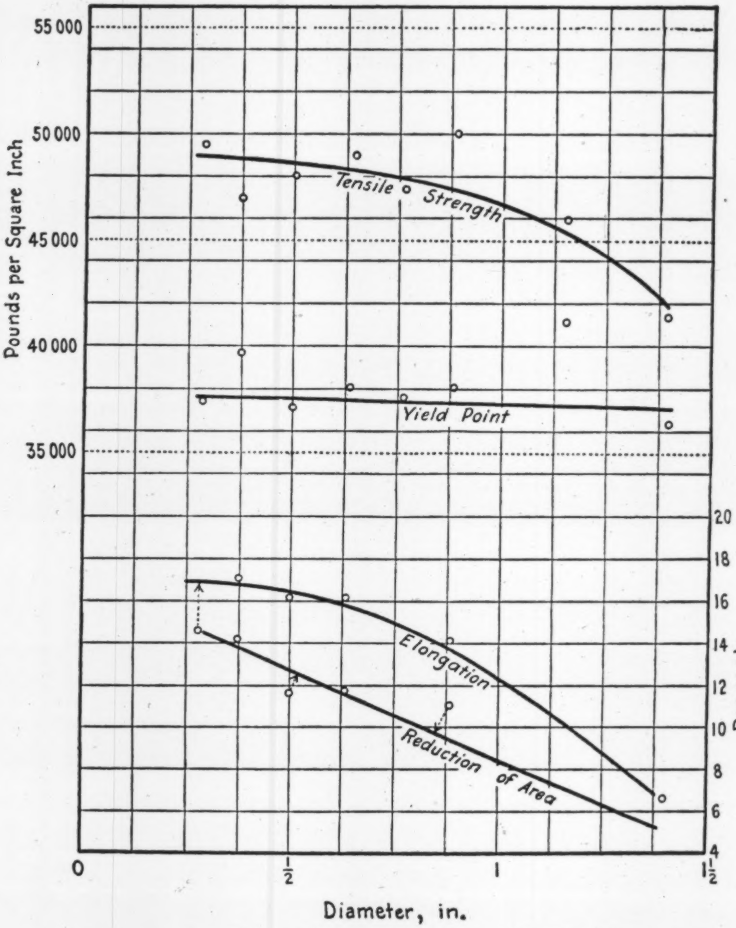


FIG. 31.—Results of Tests on Specimens Machined After Annealing.

While the exact numerical data presented hold true, of course, only for the particular metal and heat treatment involved, the following conclusions of general application may be drawn:

1. Decarburization has a favorable influence upon the strength and ductility of the product.

2. The effect of quick cooling in freezing on the surface metal of

a casting is such as to improve the strength and ductility of the product.

3. Roughness of surface of a cast specimen apparently decreases the strength and especially the ductility.

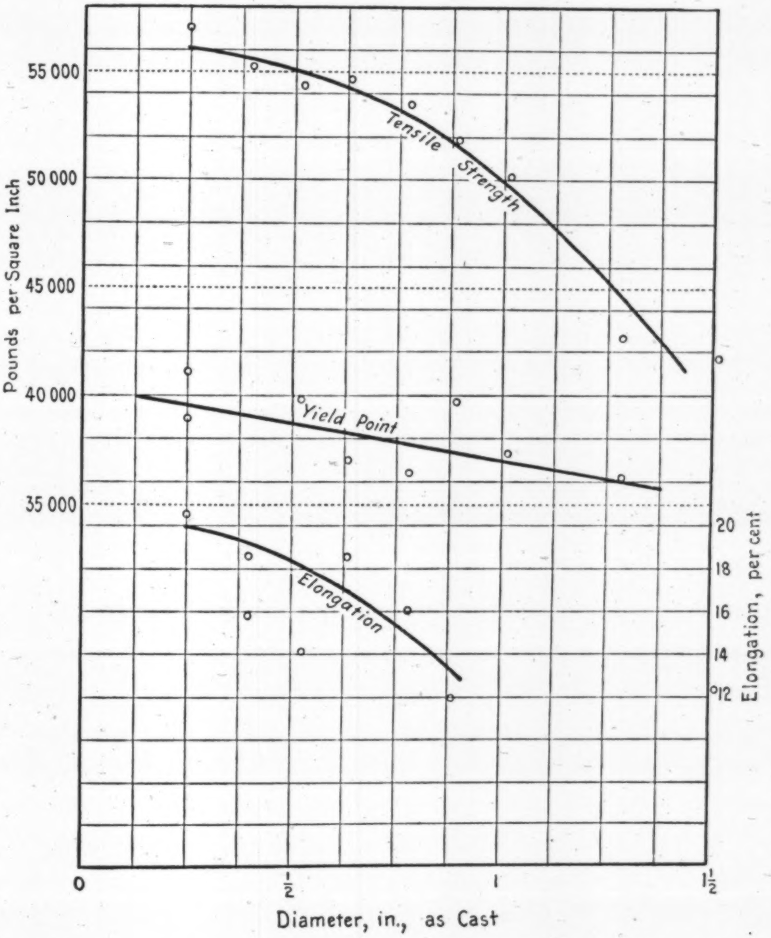


FIG. 32.—Results of Tests on Specimens Not Machined.

4. The tensile strength decreases with increasing diameter of section by an amount proportional to the cube of the diameter.

5. The elongation decreases with increasing diameter of section by an amount proportional to the $\frac{5}{2}$ power of the diameter.

6. The yield point is apparently not affected by any of the variables investigated.

A purely theoretical but rather convincing demonstration of the fallacy that the strength of malleable iron is mainly in its "skin," is to compute the requisite strength of skin metal to account for presumed loss in strength after machining. Assuming the standard $\frac{5}{8}$-in. diameter "as cast" bar, having a tensile strength of say 54,000 lb. per sq. in., to be machined to a diameter of $\frac{9}{16}$ in. (that is, a skin cut of $\frac{1}{32}$ in. removing 19 per cent of the cross-section), a 10-per-cent reduction in unit strength of the machined specimen would correspond to a skin strength of about 77,000 lb. per sq. in., and a 20-per-cent reduction would correspond to a skin strength of about 100,000 lb. per sq. in. Such strength of skin metal is of course entirely inconsistent with the material and process of manufacture.

It is not to be inferred from this discussion that the tensile properties of a specimen cut from a casting will accord with those of separately cast test bars from the same heat. As explained in the Introduction, many factors other than machining of a specimen determine the relationship between strength of casting and strength of test bar.

## V. HIGHER STRENGTH MALLEABLE IRON

As stated in the Introduction, there are certain uses of malleable iron where high strength and ductility are controlling requirements, and in recent years a grade of malleable iron having appreciably higher tensile strength, yield point and elongation than the so-called "standard specification" iron has been produced to meet these requirements.

The railway industry is one in which the higher strength irons are required for many products. Recently a large order of side stakes for gondola cars was placed with four malleable iron producers under a specification substantially higher than the standard specifications; standard test bars cast and annealed to represent each heat were required to have a yield point of not less than 35,000 lb. per sq. in. and an elongation in 2 in. of not less than 18 per cent, in addition to which the castings were subject to regular railroad inspection. Tensile strength was not specified but was determined for all bars. The records of the tension tests on the bars from all heats of the four foundries on this order (236 in number), furnished through the courtesy of the Malleable Iron Research Institute, have been analyzed and are summarized in Table IV. For each property as determined by each foundry, there are given the average and the standard deviation[1] from the average, also the maximum and minimum values reported.[1] Average values for the four foundries, giving equal weight to each individual average, are approximately:

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[1] For explanation of the term "standard deviation," see footnote 1 on p. 356.

| | |
|---|---|
| Tensile strength | 57 600 lb. per sq. in. |
| Yield point | 37 900 lb. per sq. in. |
| Elongation in 2 in | 23 per cent. |

It is of interest to compare the tensile properties of this higher strength iron with the averages for the corresponding properties as reported in Section III, and with the average tensile strength, yield point and elongation of the 5000 tests of "standard" irons given in Section IV of the Symposium. Since the last mentioned tests were reported in sufficient detail to permit of determining both the average value and the standard deviation of each property, and a similar analysis of the higher strength iron has also been made, the following comparison is possible, using the values for foundry No. 3 which are nearest the averages for strength and represent the largest number of tests:

TABLE IV.—TENSILE PROPERTIES OF HIGHER STRENGTH MALLEABLE IRON.

| Foundry | Number of Tests | Tensile Strength, lb. per sq. in. | | | | Yield Point, lb. per sq. in. | | | | Elongation in 2 in., per cent | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Maximum | Minimum | Average | Standard Deviation | Maximum | Minimum | Average | Standard Deviation | Maximum | Minimum | Average | Standard Deviation |
| No. 1..... | 55 | 65 000 | 56 000 | 58 888 | 1596 | .... | .... | .... | .... | 27.0 | 18.0 | 22.25 | 2.26 |
| No. 2..... | 63 | 59 560 | 53 000 | 57 040 | 1575 | 39 830 | 35 930 | 38 269 | 859 | 31.0 | 20.0 | 24.70 | 2.79 |
| No. 3..... | 89 | 61 710 | 55 110 | 57 689 | 1142 | 39 690 | 36 340 | 38 004 | 636 | 28.0 | 21.0 | 25.17 | 1.34 |
| No. 4..... | 29 | 61 400 | 51 380 | 56 830 | 2146 | 40 646 | 35 210 | 37 509 | 1463 | 26.6 | 18.4 | 20.26 | 1.74 |
| | | Average Tensile Strength... | | 57 612 | | Average Yield Point...... | | 37 927 | | Average Elongation | | 23.10 | |

| | AVERAGE | STANDARD DEVIATION |
|---|---|---|
| Tensile Strength, lb. per sq. in.: | | |
| "Standard" iron (5000 tests) | 54 032 | 1979 |
| Higher strength iron, Foundry No. 3 (89 tests) | 57 689 | 1142 |
| Yield Point, lb. per sq. in.: | | |
| "Standard" iron (5000 tests) | 36 605 | 1002 |
| Higher strength iron, Foundry No. 3 (89 tests) | 38 004 | 636 |
| Elongation in 2 in., per cent: | | |
| "Standard" iron (5000 tests) | 19.3 | 4.5 |
| Higher strength iron, Foundry No. 3 (89 tests) | 25.2 | 1.3 |

Since the higher strength iron was made in one plant to meet a single specification and under more closely controlled conditions, while the 5000 bars came from many heats made for a variety of purposes in different plants, the greater uniformity of results in the former as expressed by the smaller standard deviations is to be expected.

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[1] The Committee is indebted to W. A. Shewhart, Bell Telephone Laboratories, Inc., for the statistical analysis of these data.

It is also desirable to point out that the principal difference in the manufacture of a high tensile malleable iron as compared with "standard" strength iron, in respect to furnace practice, is in the use of a lower carbon content in the original furnace charge. Whereas the "standard" strength iron will average around 2.35 to 2.60 per cent carbon in the hard-iron castings, the higher strength iron will run from 2.00 to 2.20 per cent carbon. For high strength, close control of the carbon is necessary and the foundry must aim for uniformity of composition, soundness and other characteristics. During the progress of a heat, control analyses of the melt must be made with sufficient rapidity to determine the rate of oxidation of impurities and permit of any necessary additions to control composition. More skilled supervision is required throughout the entire process of manufacture.

It should also be mentioned that the tendency towards the formation of primary graphite in the hard-iron castings, which tendency is present in any portions of higher carbon castings that may have cooled slowly, as in very thick sections or in sections influenced by the insulating action of a core, is practically eliminated in the lower carbon casting, primary graphite being rarely formed in such castings even in sections of exceptional thickness. The effect of silicon, in relation to the carbon content, in controlling graphitization must of course be taken into account; this is briefly discussed in Section II on Manufacture.[1] Better control of primary graphitization, the higher strength and ductility values, and the uniformity of the product constitute the principal advantages claimed for the higher strength malleable iron by its advocates.

## VI. CUPOLA MALLEABLE IRON

Ever since the early days of the malleable iron industry, the cupola has been used for melting, although it was preceded by the crucible and small air furnace. Before the days of analyses, the air furnace had one great advantage, as a good melter could make up a heat of air-furnace malleable iron and by skillful melting and testing of the molten metal could produce a reasonably reliable grade of iron, working by "fracture." This, together with the advantages of "batch" melting, gave the air furnace a predominant position in the industry. However, there are several industries in which it has been desirable to maintain continuous operation in the malleable foundry, and this has been effected either by pouring the metal direct from the cupola, or by one of the duplexing processes. The extensive pipe fittings industry has made use of the cupola process, early taking advantage of continuous chemical control, and of such technical and metallurgical advancement as has been developed in the plants of the fittings manufacturers and by the malleable iron industry at large. These developments are still going on, and many refinements in cupola operation, coupled with close laboratory control, have brought about in well-regulated foundries a higher quality of cupola malleable iron than was obtained some years ago.

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[1] See p. 324.

## MANUFACTURE OF CUPOLA MALLEABLE IRON[1]

Raw materials are the same as those used in the air-furnace process, but the proportions vary. A typical mixture is as follows: low-phosphorus pig iron, 15 per cent; steel scrap, 35 per cent; remelt and sprues, 50 per cent. In addition, proper amounts of ferro-silicon and manganese are added. Accuracy in weighing and charging is essential.

A standard cupola is used, but unusual precautions are taken as to the charging and operation. Blast pressure is watched closely. The metal is freed of slag as it flows from the cupola and is received in a mixing ladle of the tea-pot type. The reservoir part of the ladle may be covered with a deep layer of fresh soda-ash which desulfurizes the metal to a considerable extent.

Close control is essential in making malleable iron in the cupola. Temperatures are read by optical pyrometer at 15-minute intervals. Any variation in cupola operation, height of bed, etc., is immediately reflected in the temperature of the metal, which should be held between 2775 and 2800° F. on account of the light sections to be poured. The control of chemical composition, especially carbon and silicon, is important. The proportions of these elements are indicated by the appearance of test sprues which are poured at 6-minute intervals during the entire run, and any change from proper composition can thus be taken care of immediately. Samples are sent at frequent intervals to the laboratory where rapid methods enable the metallurgist at the cupola to keep in close touch with analysis. Tension test bars are poured at regular intervals, so that a record is available of tensile properties of the metal during the progress of each day's run.

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[1] These notes on manufacture of cupola malleable iron are taken from a description, furnished to the Committee, of the procedure followed in the plant of one of the large manufacturers of malleable pipe fittings.

In many plants the molding is of the continuous type; the sand mixture is controlled, hourly tests being taken for moisture, permeability and strength. The molds are placed on conveyors, carried to the pouring platform and poured, and then conveyed to the dumping platform. One advantage of the continuous operation is that the castings are shaken out from the sand at the earliest possible moment, which effects an air quench. This tends to prevent the formation of graphite and to produce a finer grain in the hard iron. Heavier sections of cupola malleable iron are usually poured in metal molds as the quick chilling prevents formation of graphite. The same result may also be obtained by changes in composition.

The hard-iron castings are cleaned and inspected continuously, then packed in pots for annealing. The annealing process is practically the same as that previously described,[1] except that temperatures may be a little higher. Either the usual periodic type of oven is used or, if production warrants, the continuous process may be used. Close temperature control is essential. A fitting is broken from every pot in the oven as a further check on correct temperatures. The rate of cooling is held within the usual limits.

The cupola process offers advantages where continuous operation and flexibility are desired. The molten metal can be delivered at almost any required temperature and at any rate of melting. The continuous process referred to above provides large tonnage, greater turn-over of molding floor space and considerably less expenditure of labor. It is largely because of the steadiness of flow of the metal, large tonnage and good uniformity attainable, with its ease of producing great quantities of small castings of duplicate size, that the cupola is so largely used in the pipe-fittings industry.

## PROPERTIES OF CUPOLA MALLEABLE IRON

The carbon content of the white iron, before annealing, is from 2.80 to 3.10 per cent. The percentages of the usual elements present in cupola malleable iron, annealed as customary for fittings either without packing or with inert packing, will average approximately as follows:

| | |
|---|---|
| Carbon | 2.50 to 3.00 per cent |
| Silicon | 0.70 to 1.10 " " |
| Manganese | 0.40 to 0.60 " " |
| Phosphorus | under 0.20 " " |
| Sulfur | 0.15 to 0.25 " " |

Carbon is appreciably higher than is general for air-furnace iron.

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[1] See pp. 331–332.

The following tensile properties of cupola malleable iron have been furnished by three manufacturers of the product:

TENSILE PROPERTIES OF CUPOLA MALLEABLE IRON.

| | No. 1 | No. 2 | No. 3 |
|---|---|---|---|
| Tensile Strength, lb. per sq. in. | | | |
| Maximum | 58 200 | 52 000 | 45 000 |
| Minimum | 41 300 | 35 000 | 40 000 |
| Average | 49 700 | 43 000 | 43 000 |
| Yield Point, lb. per sq. in. | | | |
| Maximum | 46 800 | 37 000 | 34 000 |
| Minimum | 34 300 | 27 000 | 30 000 |
| Average | 41 000 | 33 000 | 31 000 |
| Elongation in 2 in., per cent | | | |
| Maximum | 16.0 | 12.0 | 8.0 |
| Minimum | 5.8 | 2.0 | 5.0 |
| Average | 8.1 | 7.0 | 6.5 |

The third manufacturer reports a Rockwell hardness value (B scale) of 75, averaged from 50 tests. The same manufacturer reports a test in which a bar 1 by $\frac{3}{16}$ in. in cross-section and $6\frac{1}{4}$ in. long is twisted in torsion over a gage length of $5\frac{1}{2}$ in. Twenty such bars averaged a total twist before rupture of 357 deg.

As may be seen from a comparison of the preceding table with Tables I, II and IV, cupola malleable iron has not as high strength or ductility as those of malleable irons made by other processes. However, a casting must be judged not merely on physical properties of the metal, but upon other less tangible considerations, summed up perhaps by serviceability of the casting in the use for which it is intended; and the cupola malleable casting possesses qualifications that have well adapted it to the pipe-fittings industry.

Certain advantages of the cupola in this industry inherent in the process of manufacture have been mentioned in the description of the process. Others that may be mentioned are greater fluidity and lower shrinkage, machineability, suitability for galvanizing, and tightness and strength under water or steam pressure.

Cupola malleable iron, being of somewhat higher carbon content than is customary for air-furnace and similar malleable irons, does not shrink from pattern size quite as much as the lower carbon irons, as may be seen from the relationship between carbon and shrinkage presented in Fig. 7. Also, having higher carbon, the cupola metal is somewhat more fluid and runs more freely. This greater fluidity acts to reduce porosity due to "shrinkage-sponginess."

Threading and other machining operations on cupola pipe fittings are run at high speed on production schedules and manufacturers of cupola iron (as well as some makers of duplex and other process irons) intentionally use a higher carbon with some sacrifice of tensile strength and ductility to obtain the desired machining speeds and smooth, accurate threads and surfaces. In some cases a cupola casting may be designed with slightly thicker walls to compensate for the lower physical properties of the metal.

Cupola malleable iron used in fittings is well suited for galvanizing and a large proportion of the product is so treated. There seems to be little difficulty encountered with galvanizing embrittlement of this product.

Tightness under water or steam pressure and actual bursting strength of pipe fittings are important service qualifications. The data given in Table V, reported by one of the leading manufacturers of fittings, give bursting strengths of six sizes of pipe fittings of cupola and air-furnace malleable irons.

TABLE V.—WATER-BURSTING TESTS OF PIPE FITTINGS.

All fittings were purchased in the open market as offered for sale commercially for the same types of service.

Values of pressure reported are the maximum pressures, in pounds per square inch, obtained before the fitting burst or leaked too badly to permit higher pressure.

Values reported for $\frac{1}{4}$, $\frac{3}{4}$, $1\frac{1}{4}$ and 2-in. fittings are the average of tests on one fitting in each size from each of six manufacturers of cupola fittings and from each of seven manufacturers of air-furnace fittings. For the 3- and 6-in. fittings, values reported for cupola fittings are average from four manufacturers and for air-furnace fittings from three manufacturers.

| Size of Fitting, in. | Cupola Fittings | | | | Air-Furnace Fittings | | | |
|---|---|---|---|---|---|---|---|---|
| | Ells | | Tees | | Ells | | Tees | |
| | Average Weight, lb. | Average Maximum Pressure, lb. per sq. in. | Average Weight, lb. | Average Maximum Pressure, lb. per sq. in. | Average Weight, lb. | Average Maximum Pressure, lb. per sq. in. | Average Weight, lb. | Average Maximum Pressure, lb. per sq. in. |
| $\frac{1}{4}$ | 0.104 | 8750 | 0.141 | 7600 | 0.097 | 7086 | 0.124 | 4600 |
| $\frac{3}{4}$ | 0.380 | 8833 | 0.507 | 6000 | 0.372 | 8285 | 0.482 | 7071 |
| $1\frac{1}{4}$ | 1.040 | 7000 | 1.311 | 5250 | 0.954 | 4714 | 1.200 | 4385 |
| 2 | 2.151 | 4616 | 2.757 | 3733 | 2.014 | 4528 | 2.616 | 3357 |
| 3 | ..... | .... | 6.33 | 3067 | ..... | .... | 6.94 | 4333 |
| 6 | ..... | .... | 29.35 | 2550 | ..... | .... | 27.85 | 2500 |

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## VII. THE NECESSITY FOR COOPERATION BETWEEN ENGINEER-DESIGNER AND FOUNDRY

Every effort should be made to stimulate cooperation between engineer-designer, manufacturer and foundry, for such cooperation will promote production of castings of the greatest soundness and other physical properties, at the least cost. This will be evident from a consideration of the particular defects in casting design that not only cause trouble to the founder, but make for unreliability of product and a lowering of production. Frequently the designing of castings without consideration of the foundry and metallurgical factors involved has led to high rejections and costs, where changes in design could have been made without affecting the utility of the part, which would have permitted lower cost of production and at the same time better physical properties.

The simplest form of casting, the ingot, has been given every attention by the producers of ingots for the promotion of soundness, with results that have justified the efforts. All other castings, which are in fact modified forms of ingots, present more complicated metallurgical problems in prevention of shrinkage cracks, blows, and washes than do ingots.

Some outstanding factors which should be considered by the designer of castings are:

1. The stock of the casting should be as nearly uniform as possible, avoiding direct connection of heavy sections to light sections. Shrinkage cracks will develop at points of quick change of section. Where it is necessary to vary metal from light to heavy sections, a gradual taper of the joint should be developed.

2. When heavy sections or bosses are surrounded by light sections, it is impossible to secure proper feeding of the boss, resulting in unsoundness. This trouble can be overcome by placing ribs of sufficient section on the light parts to permit feeding.

3. Cross-sections such as ribs forming a cross should at all times be avoided.

4. The design should be made sufficiently heavy or thick to permit adding a gate of such size as to insure good feeding of all sections.

5. Use as large fillets as possible and avoid sharp corners.

6. Avoid dry sand cores wherever possible, especially where castings on cooling contract around a core.

7. Parting lines should be made as even as possible.

8. Designs should be such as to provide ample draft for easy molding.

There are other considerations which will apply in many cases and it is a good rule to consult the foundry which is to produce the casting while the design is in its preliminary stage.

Some of the more important factors just mentioned will be briefly discussed.

*Gating and Heading.*—The gating and heading of castings is an art in which one can be proficient only after years of diversified experience, and while the designer is able to estimate correctly the sectional areas necessary to withstand safely the stresses to which the castings when assembled will be subjected in service, the problem as it affects both his company and the founder is beyond the mere calculation of stresses and strains and the choice of a proper safety factor.

When pattern equipment is designed and gated for metal other than malleable iron and later it is found desirable to make malleable iron castings from it, it is very often necessary to change the gating to secure sound castings. A gating arrangement for one type of metal as a rule is not satisfactory for another metal. Further, when a pattern is used for a casting of a metal other than that for which it is originally designed, changes in the design may be necessary to secure the best results.

*Provision for Adequate Thickness.*—For the sake of illustration assume that a few thin circular disk-shaped castings 4 or 5 ft. in diameter are required and that they will be subjected to very little stress in service. The designer will unquestionably be able to estimate quickly the area of section required to enable the casting to stand up safely in service, but the section as designed, while strong enough, may be so thin that only with the greatest difficulty can the casting be poured; accordingly it should be made heavier.

*Thin Patterns of Wood May Warp.*—In the example just cited, there is an additional fact to be considered that if only occasional castings are required the pattern obviously will be made of wood, so that in spite of the fact that the sections have been thickened in order to permit successful pouring of the casting, it easily may happen that the pattern is still so thin that it will warp before it is used again. Manifestly the section should be further thickened to take care of this factor, for it would be cheaper in the end to increase the section than to make a new pattern.

*Feeding Through Thin Sections to Heavier Gives Unsound Castings.*—A hub-like pattern may be very intricate, and of such a character that commercially and practically it must be gated on a flange that may be too thin as designed to permit of the use of gates thick enough to allow for proper feeding, even when feeding heads are used at available locations. A slight increase in thickness of flange would admit of the use of the proper size gates and the production of a sound casting. At the least, a local thickening of the web for the width of the gate should be permitted, provided this would not interfere with the assembling of the casting.

*Design for Sound Castings.*—In the case of malleable iron castings, soundness is of much more importance than in castings of other materials, for the reason that wherever there is a shrink there will be found a hard spot, with the result that machining difficulties will be encountered. The designer should afford the foundry every opportunity possible to secure soundness, and manifestly something is wrong with a pattern when every possible expedient is used to bring about this end and it is found to be impractical commercially.

*Square Corners Should be Avoided.*—The outer circumference of thin flanges should not be made with square corners if this can be avoided. It must be borne in mind that the castings as initially cast consist of brittle white iron and are in a state of strain; so that a sharp corner may readily develop a minute check that will extend into a crack, particularly in the vicinity of the gates where the metal cools last. In cases of this kind, losses due to cracks will be greatly lessened if the corners are slightly rounded off.

*Advantage of Uniform Section Thickness.*—The designer frequently ignores the fact that a thin section will cool in the mold much faster than one that is heavier, which in turn means that the metal in each is passing through its critical temperature range at different intervals of time, so that one section is contracting while another is expanding. The strains thereby set up may cause the foundry a prohibitory loss due to cracks. While disproportionate sections are impossible to avoid, it frequently is possible to even up section thicknesses by making the heavy one slightly thinner and the lighter one slightly heavier, giving in the end a stronger casting without increase in weight. The designer should realize that frequently a casting actually is made up of various parts, each of which consists of a separate casting joined to its neighbor by a connecting section of metal. The thin section cooling more quickly will draw metal from an adjacent heavy section and porosity of metal, if not a serious shrink, may take place in a very vital location.

The bane of the founder's life is the pattern of disproportionate sections; and while it is realized that this condition is sometimes unavoidable, it frequently happens that the situation could be improved greatly through certain modifications that not only would serve to make the casting more reliable, but facilitate the work in the foundry.

Generous fillets should be used where possible and sharp corners avoided, the latter being not only a source of weakness, but preventing proper cleaning of the casting.

*Dry Sand Cores.*—When practical, castings should be so designed as to avoid the use of dry sand cores. Even those that are made of the weakest mixture will offer much greater resistance to the iron as it starts to contract after solidification has started than a core of green sand which will collapse when subjected to the slightest pressure. There is a certain narrow range of temperature during solidification within which white iron is extremely tender, and if on passing through this range the iron meets resistance the loss due to cracks will be very heavy, especially if the core forms the interior surfaces of the casting. The situation is not so serious in other cases, as for instance where a core is used to form a projection on an exterior surface.

*Faulty Design Delays Production.*—Complaints frequently are made that it takes the foundry too long a time to submit a sample casting from a pattern furnished. When this occurs, it will be found generally that the design is so faulty in some of the details mentioned, that many trials have been made as to gating and other factors before some suitable method has been found that will insure freedom from shrink on the one hand and a prohibitory loss due to cracks, etc., on the other.

Through cooperative effort better castings will be secured; there will result a saving of time to both manufacturer and foundry and unfortunate misunderstandings frequently will be avoided.

## VIII. RÉSUMÉ OF CURRENT SPECIFICATIONS FOR MALLEABLE IRON CASTINGS

There are current today a number of specifications for malleable iron castings, promulgated for the most part by consumers of such castings, especially the railroads. The National Directory of Commodity Specifications published by the U. S. Department of Commerce in 1925 lists 41 specifications for general malleable castings, of which 25 are specifications of railroads and railway associations, 11 are specifications of other consumers (including the National Government), and 5 are those of national technical societies. In an endeavor to give a concise picture of the present status of specifications for malleable iron castings, Table VI has been prepared, listing b the physical properties required in the specifications published by a number of the technical societies and by a few typical consumers and manufacturers. Where revisions of the specifications are known to be under consideration, available information thereon is given in the last column of the table.

On the whole, there is pretty general agreement on desired minimum tensile properties, with the trend for higher strength and ductility values for railroad castings reflected by the specifications

## TABLE VI.—SUMMARY OF PHYSICAL PROPERTIES SPECIFIED IN CURRENT SPECIFICATIONS FOR MALLEABLE IRON CASTINGS.

Unless otherwise noted, the specifications covered in this summary require that malleable iron castings shall be produced by either the air-furnace, open-hearth or electric-furnace process.

| Name of Organization | Date of Adoption (or latest revision) of Specification now Effective | Minimum Physical Properties Specified | | | Remarks |
|---|---|---|---|---|---|
| | | Tensile Strength, lb. per sq. in. | Yield Point, lb. per sq. in. | Elongation in 2 in., per cent | |
| American Society for Testing Materials. | 1930 | 50 000 | 32 500 | 10 | Specification A 47 - 30. A new grade of castings has been proposed by the Society's committee on malleable castings for addition to Specification A 47, having minimum tensile strength 53,000 lb. per sq. in., minimum yield point 35,000 lb. per sq. in., minimum elongation in 2 in., 18 per cent. Action on this proposal will be taken by the Society in June, 1932. The same committee is also developing specifications for cupola malleable iron. |
| American Foundrymen's Assn....... | .... | ..... | ..... | .... | Does not publish specifications for malleable iron. Officially cooperates in development of A.S.T.M. specifications. Has indorsed A.S.T.M. Specification A 47. |
| American Railway Assn............. | 1928 | 50 000 | 30 000 | 15 | Has under consideration a recommendation of the Malleable Iron Research Institute suggesting that the physical properties be somewhat increased. |
| Society of Automotive Engineers.... | 1922 | 45 000 | ..... | 7.5 | Has under consideration revision of present specification to bring it into conformity with A.S.T.M. Specification A 47 - 30. |
| American Society of Mechanical Engineers........... | 1931 | 50 000 | 32 500 | 10 | A.S.M.E. Boiler Code. Specifications identical with A.S.T.M. Specification A 47 - 30. |
| American Bureau of Shipping......... | .... | ..... | ..... | .... | Does not embody specifications for malleable iron in its Rules for Building and Classing Steel Vessels. When called on to furnish certificate of test for approval of quality, requirements of latest A.S.T.M. specification are used. |
| Federal Specifications Board....... | 1928 | 50 000 | ..... | 10 | Specification QQ-I-666. Requires also minimum Charpy impact value of 7.0 ft-lb. and minimum Izod value of 5.0 ft-lb. for galvanized castings. |
| American Marine Standards Committee........... | .... | ..... | ..... | .... | Has not formulated definite requirements for malleable castings. In its standard, Cleats for Ships, provides that malleable castings shall meet requirements of A.S.T.M. specification. |
| Manufacturers Standardization Society of the Valve and Fittings Industry........... | 1931 | 40 000 | 30 000 | 5 | Specification No. SP-22-1931, entitled "Malleable Iron Castings for Flanges, Pipe Fittings and Valve Parts." Covers castings produced by the cupola, air-furnace, open-hearth, electric-furnace or other approved process. |
| Malleable Iron Research Inst....... | 1931 | 50 000 | 33 000 | 12.5 | Has also adopted specifications for malleable castings for railroad work with minimum yield point 35,000 lb. per sq. in. and minimum elongation in 2 in., 18 per cent, recommending that A.S.T.M. and A.R.A. adopt similar specifications. |
| General Electric Co.. | 1924 | 50 000 | .... | 10 | These requirements are those adopted as standard by A.S.T.M. in 1924. A new edition of the specifications is being prepared in which it is expected to embody the provisions of A.S.T.M. Specification A 47 - 30. |

TABLE VI (*Continued*).—SUMMARY OF PHYSICAL PROPERTIES SPECIFIED IN CURRENT SPECIFICATIONS FOR MALLEABLE IRON CASTINGS

| Name of Organization | Date of Adoption (or latest revision) of Specification now Effective | Minimum Physical Properties Specified | | | Remarks |
|---|---|---|---|---|---|
| | | Tensile Strength, lb. per sq. in. | Yield Point, lb. per sq. in. | Elongation in 2 in., per cent | |
| Westinghouse Electric and Manufacturing Co......... | 1924 | 50 000 | ..... | 10 | P.D. Specification No. 2051 C. |
| Electric Bond and Share Co......... | 1926 | 50 000 | 30 000 | 10 | Also requires impact test for galvanized specimens to show impact values not less than values for plain (control) specimens. |
| General Motors Co.. | .... | ..... | ..... | .... | Does not have definite physical properties specification. Integral test lugs must show adequate ductility and freedom from flake graphite, as determined by fracture. Castings must show satisfactory machineability. |
| International Harvester Co......... | 1922 | 48 000 | ..... | 10 | Also requires bend test of ½-in. round bar, through minimum angle of 100 deg. |
| New York Central Lines............ | 1924 | 50 000 | 30 000 | 15 | Specification No. 1086-A. |
| Pennsylvania Railroad Co.......... | 1931 | [a] | 35 000 | 18 | Specification No. 164, issued November, 1931. |

[a] Recorded as Information.

of the American Railway Association, the two railroad lines quoted, and the proposals under consideration in the American Society for Testing Materials and the Malleable Iron Research Institute. The influence of the current A.S.T.M. Specifications for Malleable Iron Castings (A.S.T.M. Designation: A 47 – 30)[1] on development of specifications by other organizations is evident from the notes in the table. Furthermore, other provisions of specifications for malleable iron, not here tabulated, such as test specimens, methods of testing, special tests, number of tests, retests, reannealing, workmanship, finish and inspection, follow very generally—in many instances identically—the provisions of the A.S.T.M. standard.

Specifications were adopted this year by the Manufacturers Standardization Society of the Valve and Fittings Industry covering malleable castings for flanges, pipe fittings and valve parts made by the cupola process, as well as by other processes. Specifications for cupola malleable iron are also under development in the American Society for Testing Materials.

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[1] 1930 Book of A.S.T.M. Standards, Part I, p. 491.

# DISCUSSION, SYMPOSIUM ON MALLEABLE IRON CASTINGS

Mr. Leo J. Kelly[1] (*presented in written form*).—Our company prepared, in a brief way, some data for inclusion in the Symposium. In reading the preprint we find some points that we believe need revision before the Symposium is finally published. I am presenting these, not in a spirit of criticism, but simply to give them proper emphasis. I want to congratulate the American Society for Testing Materials, the American Foundrymen's Association and the members of the Symposium Committee for their efforts in making possible this valuable contribution to the literature on malleable iron.

On page 353, the subject of galvanizing embrittlement is referred to and the following statement is made: "It is known, however, that the tendency to this embrittlement is related to chemical composition to such a degree that the fault can be avoided by chemical means." I am not in accord with this statement. I have not yet seen authoritative proof of the statement; while on the other hand, I have found by actual tests that ordinary malleable iron of any composition within commonly accepted limits, when heated to 850° F. and rapidly cooled, will become embrittled. I would suggest therefore that the statement be omitted or modified.

On page 370, in discussing the high tensile malleable iron, the following statement appears: "For high strength, close control of the carbon is necessary and the foundry must aim for uniformity of composition, soundness and other characteristics." I should like to point out that malleable iron of any grade to be at all acceptable should be just as sound and free from shrinks and as uniform as the high strength product. It is entirely misleading to assume that freedom from ordinary troubles and defects is an attribute of either an average or a very low tensile product. A closer control of carbon is essential because of a rigid guarantee, whereas in the weaker product a considerable variation in carbon means little by reason of the liberal specifications under which it is made.

In the same section of the Symposium, on page 370, the following statement appears: "Better control of primary graphitization, the higher strength and ductility values, and the uniformity of the product constitute the principal advantages claimed for the higher strength malleable iron by its advocates." I believe the words "claimed for" and "by its advocates" should be stricken out and the sentence corrected to read, "Better control of primary graphitization, the higher strength and ductility values, and the uniformity of product constitute the principal advantages of higher strength malleable iron." Actually, it has been fully demonstrated that higher strength malleable iron has all the advantages of lower strength iron and none of its disadvantages.

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[1] Superintendent, Fort Pitt Malleable Castings Co., Pittsburgh, Pa.

In discussing cupola malleable iron in Section VI, the statement is made on page 374 that: "Cupola malleable iron used in fittings is well suited for galvanizing and a large proportion of the product is so treated." This implies that malleable iron produced by other dependable processes is not so well fitted for galvanizing. We have as yet to see reliable proof to substantiate this statement and therefore feel that this implication should not be made.

MESSRS. F. L. WOLF[1] AND L. A. MEISSE[2] (*presented in written form by Mr. Meisse*).—Laudable work has been done by the Symposium Committee in collecting so much specific data on malleable iron. The work will be welcomed by producers and consumers alike. It will be regarded as another big step ahead in the endeavor to clear up some of the mysteries with which some of the manufacturers in the past have surrounded their product and methods of manufacture, and give therewith a better appreciation to the engineer and designer of the essential characteristics of this type of founded iron. It cannot help but stimulate a broader application of this material.

The following comments are offered regarding the physical and chemical data. It appears to us that the average physical properties of malleable iron as given on page 335 are higher than could be expected to represent the average for the industry. While these physical properties are representative of what is usually termed high-strength malleable, which is made chiefly by the members of the Malleable Iron Research Institute, it appears doubtful whether the average for the malleable iron industry as a whole is as high as given. It is therefore suggested that the canvass for average physical properties, minimum and maximum, of a larger number of malleable iron manufacturers be continued. The physical properties given in the Symposium are taken from averages of 20 manufacturers, whose names are given in the preface, whereas there are from 120 to 150 manufacturers in this country. This would give an opportunity for revision of these data.

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[1] Chief Engineer and Technical Superintendent, The Ohio Brass Co., Mansfield, Ohio.

[2] Research Metallurgist, The Ohio Brass Co., Mansfield, Ohio.

Furthermore, if the average elongation in 2 in. is given as 18 per cent, a minimum specification of 18 per cent for elongation as representative of the quality of malleable iron made today appears to the writer, from the information given on page 354, to exclude about 35 per cent of the iron tabulated.

On page 353 the statement is made: "It is known, however, that the tendency to this embrittlement is related to chemical composition to such a degree that the fault can be avoided by chemical means." The question we should like to raise is, "Who does control it that way?" In order to have a composition free from embrittlement it is necessary to keep the phosphorus content lower than 0.15 per cent and the silicon content lower than 0.80 per cent, and such percentages of silicon are found only in malleable iron of the high-carbon type, not referred to in this Symposium.

On page 336 it is stated that: "Inasmuch as there is no combined carbon in malleable iron, phosphorus may be present in the amount indicated (0.20 per cent) without danger of cold shortness." It is not quite clear what is meant by "cold shortness." If this means that the material is brittle at and below room temperature, this statement can be accepted only with reservations. A phosphorus content of 0.20 per cent and a silicon content as high as 1.10 per cent, as mentioned in the analyses, will in most cases, but not always, give brittle iron, a condition which is not of the type referred to as insufficiently annealed but which is essentially the same as found after galvanizing.

In discussing the packing material on page 331, the use of iron oxides in various forms is mentioned. While the use of iron oxides comprised the original Reaumuir process, the present practice where a high degree of machineability is essential precludes the use of iron oxide because it is responsible for decarburization of the rim and sometimes of the whole section. After examining a fairly large variety of representative malleable iron, we find that the majority of iron does contain a more or less decarburized rim. Whether this is intentional or not is debatable. It affects machineability unfavorably, and also it is evident that the addition of mill scale or iron oxide in some form or other to the packing material promotes this condition and cannot, therefore, be considered as a desirable practice where machineability is of importance. Repeatedly, we have found malleable iron of the high-strength type to contain severely decarburized rims and it appears that in these cases mill scale or iron oxide has been purposely added to the packing in order to boost the strength, but of course accomplishes this only by sacrificing machineability.

The photomicrographs given on page 326, while perhaps not doing full justice to the original photographs, merit some comment. It is not clear why the material in Fig. 4 is supposed to contain more carbon and therefore be less suitable for production of good malleable iron than that in Fig. 1, unless some primary graphite visible under the microscope has not been reproduced. The difference between Figs. 4 and 1 would be a larger grain size in the former, which would have no bearing either on the carbon content or on the final quality of the iron in the annealed state, as we see it.

In closing we want to emphasize that our criticisms are offered purely for the sake of assisting to clear up some points which might be misinterpreted, and in no way to detract from the inherent value of the Symposium.

MR. H. A. SCHWARTZ.[1]—Mr. Kelly's statement that there was no proof of ability to avoid embrittlement by chemical means was answered by Messrs. Wolf and Meisse. The thing has been demonstrated to the satisfaction of the authors and apparently to the satisfaction of other people, but what Mr. Meisse says as to whether or not the results are made use of in practice is another matter—the statement is that the phenomenon can be controlled, not that it is controlled.

MR. KELLY.—I understand the statement regarding chemical means of avoiding embrittlement in the Symposium is based on Bean's demonstration that low-phosphorus low-silicon malleable iron is immune to galvanizing embrittlement. I would simply say that our findings are at variance with those of Bean.

During the past year we decided to investigate the claim that low phosphorus and low silicon serve to inhibit galvanizing embrittlement. Since our own product is of average phosphorus content and high rather than low in silicon, we cut test pieces from assorted malleable iron scrap in our possession, selecting pieces that were found to be unusually low in both phosphorus and silicon, phosphorus averaging from 0.116 to 0.15 per cent and silicon from 0.65 to 0.79 per cent. The results were contrary to the findings reported by Bean, for the pieces selected proved to be susceptible to galvanizing embrittlement.

THE SYMPOSIUM COMMITTEE *(closure to discussion, by letter)*.—The Committee deems it expedient to make formal response to certain points brought up in the discussion of the Symposium.

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[1] Manager of Research, National Malleable and Steel Castings Co., Cleveland, Ohio.

The work of Bean, for which a specific reference has been included in the text at the appropriate place, is believed to furnish sufficient substantiation for the statement as to the chemical prevention of galvanizing embrittlement.

Lacking any details of Mr. Kelly's observations to the contrary, it is impossible critically to evaluate the relative merits of Bean's and Kelly's conclusions. It is suggested that the latter seek publication of his complete observations and conclusions in some appropriate place. Qualified persons will then have opportunity to decide the question to their own satisfaction or perhaps to explain the cause of any differences.

The sentence on page 370 reading "Better control of primary graphitization, the higher strength and ductility values, and the uniformity of the product constitute the principal advantages claimed for the higher strength malleable iron by its advocates," is a correct statement of the Committee's viewpoint and does not in its judgment require modification.

In this case as also in the statement "Cupola malleable iron used in fittings is well suited for galvanizing and a large proportion of the product is so treated," page 374, the writers meant neither more nor less than they said, and those seeking to draw further implied meanings can not hold the Committee responsible for their conclusions. Certainly it is a matter of common knowledge that a large proportion of the malleable pipe fittings produced are galvanized, and presumably successfully.

The suggestion by Messrs. Wolf and Meisse that the physical data represent chiefly the product of members of the Malleable Iron Research Institute and hence are not representative of the range of values in the industry, is not easy to discuss. At least two of the seventeen sources of information given in the Symposium preprint covered a considerable number of foundries, so that while the seventeen may appear a rather small proportion of the total number of foundries making malleable iron in the country, the distribution is actually relatively wide. Note 3 has been added to Table I recording the fact that 27 sources of data are represented in the reports of two of the seventeen investigators. Attention is also directed to the fact that the data reported in Table II came from 52 foundries.

The averages shown in Table I are not weighted averages, but calculation showed that there was no great difference between the result of weighted and numerical averages; although if we study the table closely we can probably note differences of practice. In spite of the criticism that the table does not represent a true average, it probably comes very near to doing so.

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The Committee made diligent efforts to procure data from certain important interests whose omission seemed conspicuous and who were thought to look favorably on rather lower strength iron. It also included in its original inquiries as complete a list of producers as was available. Any lack of representativeness of data is then due to lack of response by the desired sources and certainly not to any intentional selection by the compilers.

It is perhaps unavoidable that data should be more readily available from large producers and trade organizations than from scattered smaller units. On the other hand a weighting by tonnage would probably show that only one single producer is missing whose production is a major factor in the industry, always excluding cupola malleable which is separately considered.

Messrs. Wolf and Meisse seem under a misapprehension in using the data on page 354 as a basis for argument regarding the desirability of an 18-per-cent elongation requirement in a specification. The data tabulated on page 354 are for iron produced to a 10-per-cent elongation specification, as stated on page 358, and it is nowhere maintained that an 18-per-cent elongation would not require departures in practice from that used under the present specifications. An entire section, V, of the Symposium is devoted to the question of these higher strength irons, pointing to some of these differences. The entire question of the adoption of such higher specifications for general purposes hinges on differences of opinion as to the results, in other directions, of such changes of practice.

If the purpose of the critics is merely to point out that the present commercial product frequently falls below the higher specification, then their conclusion is so obviously correct as to require no comment.

The Committee nowhere advocates the average properties given on page 335 as a basis of specification and has modified the text to make this definitely clear.

The danger of combined high phosphorus and silicon is recognized, although we scarcely know whether the resulting difficulty is to be ascribed to phosphorus or silicon or heat treatment, for it can be overcome by heat treatment. It was desired to point out to engineers a difference between malleable castings and steel products which logically explains why phosphorus in steel is properly covered by much lower maximum limits in specifications than are customarily found in very good malleable iron.

With reference to the photomicrographs shown in Figs. 1 and 4, the following further explanation is offered. The carbon content of the specimen whose micrograph is shown in Fig. 1 was 2.05 per cent, that shown in Fig. 4, 2.83 per cent. This specimen was secured from a piece knocked from a large white iron casting. The individual submitting the photomicrographs feels that Fig. 4 also shows metallographic evidence of a higher carbon content. The structural conditions existing in the hard iron determine in large measure the conditions that will exist in the annealed product. In a given area in Fig. 1 there are many more grains of pearlite and free cementite than in Fig. 4 and this condition will be directly reflected in the annealed product.

# AN INVESTIGATION OF METHODS TO DETERMINE THE MACHINEABILITY OF MALLEABLE IRON CASTINGS

By O. W. Boston[1]

## Synopsis

As a preliminary step in investigating the machineability of malleable iron castings for the Malleable Iron Research Institute, a study was made to determine what manner of metal cutting or other types of tests were best adapted to portray their characteristics. Specially designed castings having sections $\frac{1}{4}$, $\frac{3}{4}$, and $\frac{1}{8}$ in. thick, respectively, were made and annealed.

Cutting characteristics at different depths of the decarburized skin were determined for each thickness of the castings submitted by several different methods, such as the force on a planer tool of the cut-off type; the energy required to remove a chip by milling; the torque, thrust, and net power developed by a counterboring tool. In addition to the above, the torque, thrust, and power developed by a twist drill $\frac{3}{4}$ in. in diameter were determined for the interior of sections of different thicknesses. The time for a drill $\frac{1}{4}$ in. in diameter to penetrate 0.100 in. under a given feed load was determined for the skin and body of each section.

Photomicrographs were prepared showing the structure from the surface of the metal to some distance into the interior, and total-carbon contents for very thin sections at different distances from the surface also were found. Hardness tests were determined by the Vickers machine on an inclined plane extending from the surface to the interior metal. An attempt is made to correlate these various factors.

It is found that the outside surfaces of each of the four castings were decarburized to a depth of approximately $\frac{1}{8}$ in. The total carbon decreases perceptibly from this distance to the surface. The immediate surface consists of ferrite, below which is a layer of pearlite. At still greater depth the graphitic carbon structure is found. All methods of tests indicate the surface of the casting to machine with comparative ease. Between 0.015 and 0.050 in. beneath the surface, however, the metal machines with greatest difficulty. At a depth of approximately $\frac{1}{8}$ in., the metal machines with the greatest ease. Below this point the machining characteristics appear to be fairly constant. The skin on a thick section machines with greater difficulty than the skin on a thin section. The body in the thick section machines with more difficulty than the body of the thin section. It appears that improved machining conditions could be obtained by annealing the castings in such a way as to reduce or eliminate the surface decarburization.

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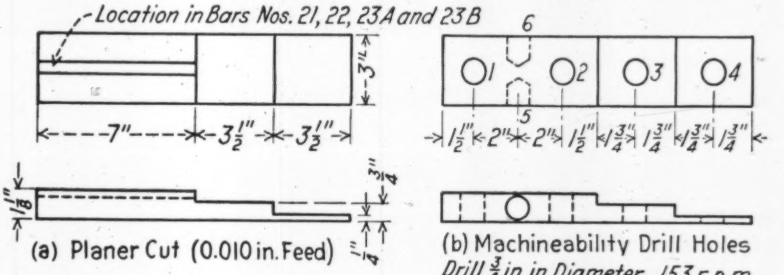
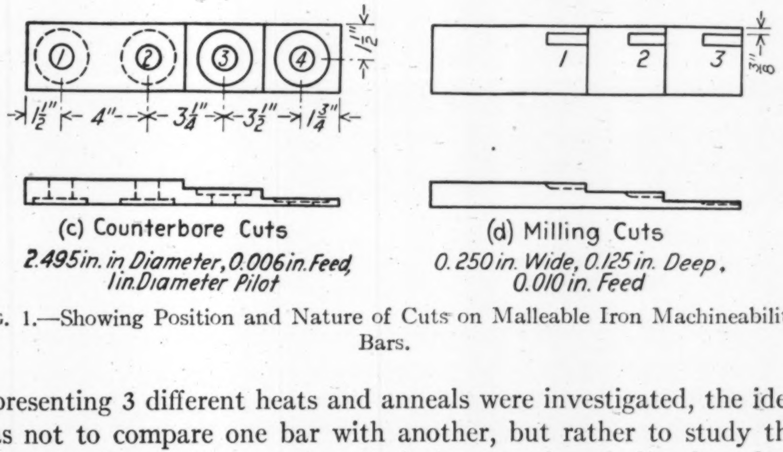
[1] Professor, College of Engineering, University of Michigan, Ann Arbor, Mich.

This paper summarizes the results of a series of carefully executed laboratory experiments on malleable iron castings designed especially for the purpose. The results of a study to learn what manner of metal cutting is best to portray the characteristics of malleable iron castings with particular reference to the decarburized skin are presented. These machining properties are correlated with the results of hardness tests, chemical analyses, and structures determined from photomicrographs. While four similar machineability bars representing 3 different heats and anneals were investigated, the idea was not to compare one bar with another, but rather to study the influence of annealing on the physical properties of the given bar. The cutting characteristics of thin sections of the decarburized skin at different depths are determined for each of the bars.



(a) Planer Cut (0.010 in. Feed)

(b) Machineability Drill Holes
Drill ¾ in. in Diameter, 153 r.p.m., 0.012 in. Feed



(c) Counterbore Cuts
2.495 in. in Diameter, 0.006 in. Feed, 1 in. Diameter Pilot

(d) Milling Cuts
0.250 in. Wide, 0.125 in. Deep, 0.010 in. Feed

FIG. 1.—Showing Position and Nature of Cuts on Malleable Iron Machineability Bars.

## THE MATERIALS TESTED

The materials on which the preliminary machining tests were conducted consisted of four machineability bars 3 in. wide and 14 in. long, but stepped so as to provide thicknesses of 1⅛ in. for a length of 7 in., ¾ in. for a length of 3½ in., and ¼ in. for a length of 3½ in., as shown in Fig. 1. These bars are numbered 21, 22, 23A, and 23B and are listed with their complete chemical analyses in Table I. The complete chemical analyses were determined from thin flat chips taken between 0.190 and 0.200 in. from the surface in the $1\frac{1}{8}$-in. thick section. This depth was selected inasmuch as it was found to be in a portion of the bar of uniform quality and not influenced by the variable analysis of the decarburized skin. The complete planer-tool chip at selected depths was analyzed.

Bar No. 21 was produced and annealed by one manufacturer, while bars Nos. 22, 23A, and 23B were produced and annealed by a second manufacturer. The annealing cycle for bar No. 21 was as follows: from 500 to 1690° F. in 35 hours, held at 1690° F. for 38 hours, and then cooled to 1040° F. in 81 hours. The bar was packed in mill scale and annealed in a hand-fired coal-burning furnace.

Bars Nos. 22, 23A and 23B were annealed in a small oil-fired oven. They were packed in a pot with furnace slag. It required 18 hours to bring the oven up to 1600° F., at which temperature it was held for 44 hours. It was then cooled down in 10 hours. Bars Nos. 23A and 23B are supposedly from the same heat and the same anneal. Inasmuch as both were tested, one was designated as A and the other as B.

Table I.—Chemical Analyses of Malleable Iron Machineability Bars Determined from Chips at 0.190 to 0.200 in. from Surface in the $1\frac{1}{8}$-in. Section.

| Bar | Total Carbon, per cent | Graphitic Carbon, per cent | Manganese, per cent | Silicon, per cent | Sulfur, per cent | Phosphorus, per cent |
|---|---|---|---|---|---|---|
| No. 23B | 1.984 | 1.907 | 0.250 | 1.13 | 0.140 | 0.162 |
| No. 23A | 2.012 | 1.954 | 0.250 | 1.01 | 0.097 | 0.166 |
| No. 22 | 2.004 | 1.949 | 0.260 | 1.13 | 0.120 | 0.145 |
| No. 21 | 2.191 | 2.168 | 0.260 | 0.87 | 0.089 | 0.146 |

The chemical analysis of the hard iron as furnished by the foundries is as follows:

| | Total Carbon, per cent | Silicon, per cent | Manganese, per cent |
|---|---|---|---|
| Bar No. 21 | 2.32 | 0.87 | 0.24 |
| Bar No. 22 | 2.38 | 1.04 | 0.23 |
| Bar No. 23 (A and B) | 2.26 | 1.01 | 0.25 |

There is a difference in the silicon content of bar No. 22 from 1.04 per cent in the hard iron, as determined by the foundry, to 1.13 per cent in the annealed iron, as determined in the University laboratory. This difference may be due to an error in the analysis. The manganese in the hard iron is given as 0.23 per cent, whereas it is 0.26 in the annealed iron. The analysis of the annealed bar

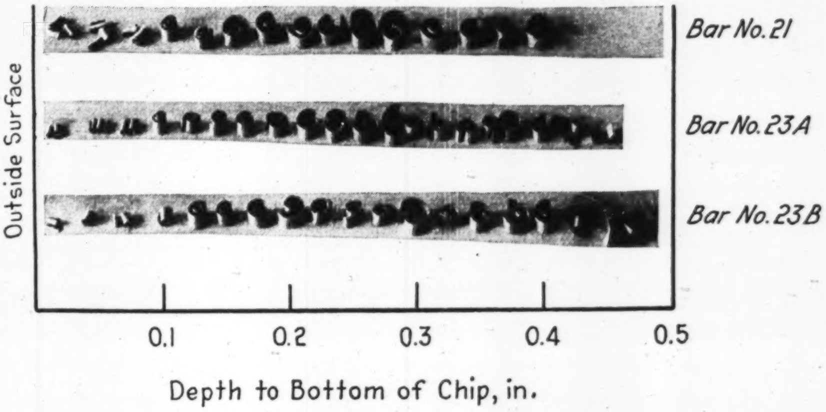

Outside Surface
Bar No. 21
Bar No. 23A
Bar No. 23B
0.1
0.2
0.3
0.4
0.5
Depth to Bottom of Chip, in.

FIG. 2.—Planer-Tool Chips from Bars Nos. 21, 23A, and 23B, ½ in. Wide and 0.010 in. Thick, Taken from the Surface at the Left to a Total Depth of ½ in.

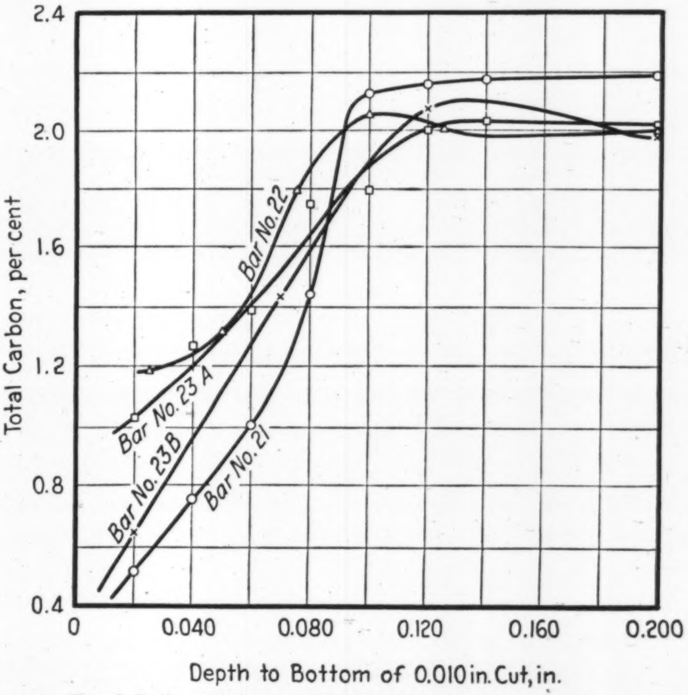


FIG. 3.—Total Carbon Content of Malleable Iron Bars as a Function of the Depth from the Surface.

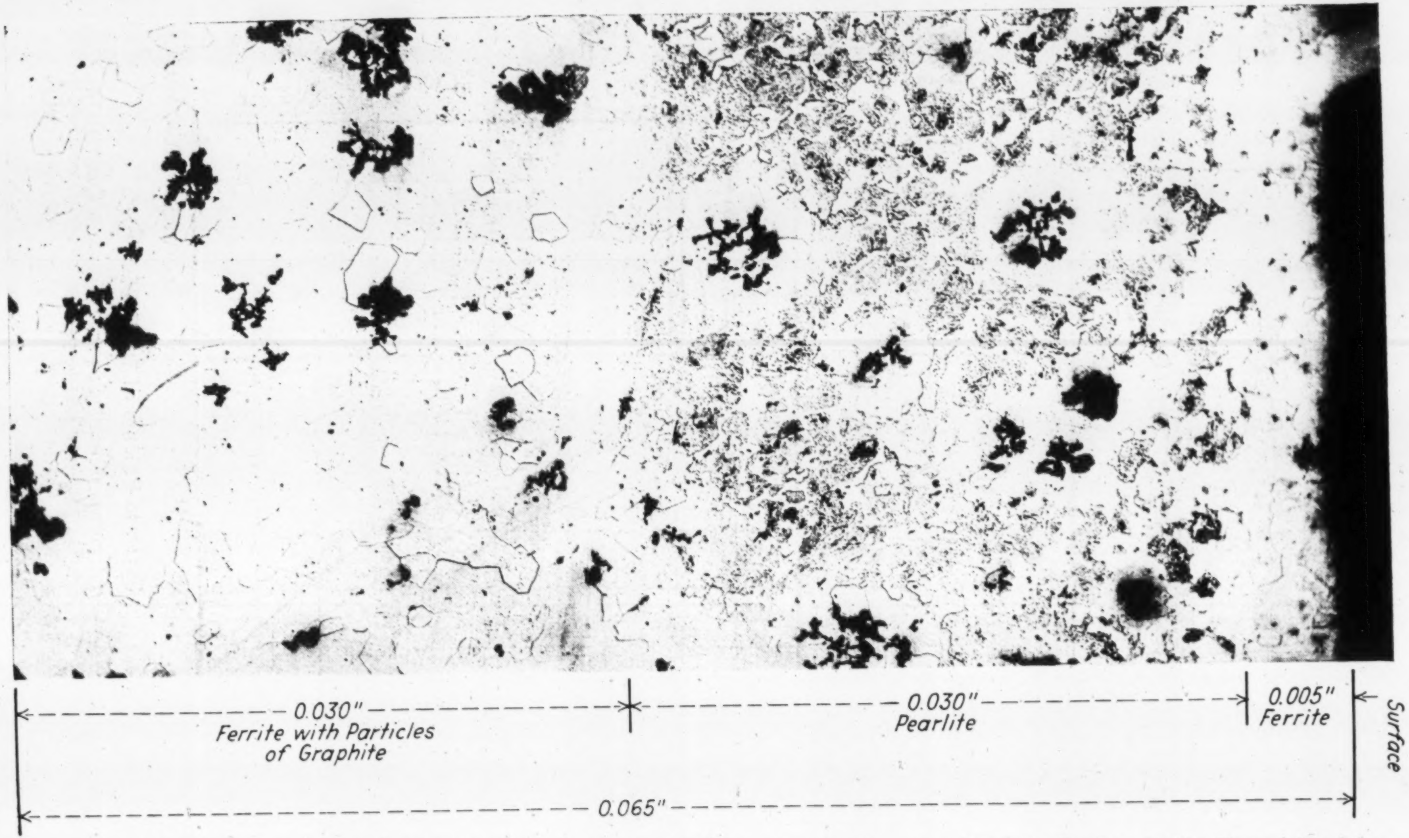

Surface
0.005"
Ferrite
0.030"
Pearlite
0.030"
Ferrite with Particles of Graphite
0.065"

FIG. 4.—Photomicrograph of Malleable Iron Bar No. 23B (× 100), Showing the Structure from the Surface at the Top of the ¼-in. Thick Section. (Etched with 4 per cent Nital for 4 to 5 Seconds.)

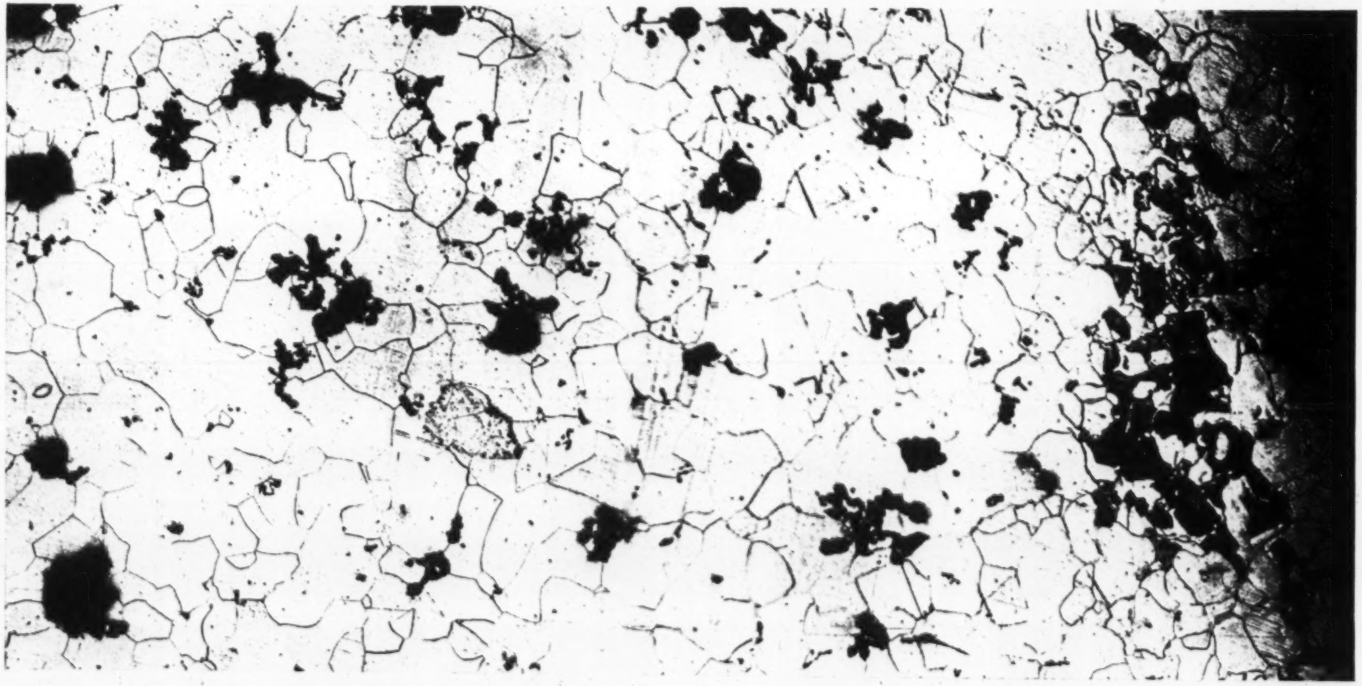


FIG. 5.—Photomicrographs of Malleable Iron Bar No. 23B (× 100), Showing the Structure from the Surface at the Top of the ¾-in. Section. (Etched with 4 per cent Nital for 4 to 5 Seconds.)

No. 23A corresponds with the hardness of the hard iron for silicon and manganese, whereas bar No. 23B, supposedly the same as No. 23A, shows a silicon content 0.12 per cent higher and a sulfur content 0.044 per cent higher.

In connection with the machining tests on the planer, planer-tool chips ½ in. wide and 0.010 in. thick were removed successively, each successive chip being at the bottom of the preceding one. These planer chips for each successive cut from the surface are shown in Fig. 2 for bars Nos. 21, 23A, and 23B. The chips correspond to the points indicated for total carbon content of the respective bars in Fig. 3.

These planer-tool chips were submitted to total-carbon analyses with the results shown graphically in Fig. 3. This shows, for example, that the total carbon in bar No. 21 in a chip taken between 0.010 and 0.020 in. from the surface is 0.516 per cent. The total-carbon content increases with increased depth from the surface to 2.133 per cent in the chip taken from 0.090 to 0.100 in. depth. There is only a slight additional increase in total-carbon content for further depths until the total carbon reaches 2.191 per cent in the chip from between 0.190 and 0.200 in. There appears to be less decarburization for the other bars. A maximum total-carbon content is reached for bars Nos. 21 and 22 at 0.100 in. depth, while the maximum total-carbon content for bars Nos. 23A and 23B is reached at a depth of 0.120 in. The four curves appear to be quite similar.

Two photomicrographs are shown in Figs. 4 and 5 for bar No. 23B. The photomicrograph shown in Fig. 4 was taken from the transverse face of a specimen, ⅜ in. from the center of the side of the ¼-in. section. That shown in Fig. 5 is taken from the transverse face of a specimen, ½ in. from the center of the side of the ¾-in. thick section. Figure 4 shows the ¼-in. section with the edge or outer surface of the bar at the top. From the portion visible there appears to be a thin layer of ferrite having an actual thickness of 0.005 in., in which there is practically no carbon. Below this layer is a layer of pearlite for approximately 0.030 in. Below the pearlite there appears an area of ferrite interspaced irregularly with particles of graphite. An examination showed this same condition to continue further to a considerable depth. Figure 4 actually shows the structure to a total depth from the surface not to exceed 0.065 in. Figure 5 shows the structure from the surface at the top of the ¾-in. section. A structure similar to that of Fig. 4 is seen with perhaps a more dense and concentrated portion of pearlite.

Neither of these photomicrographs was considered satisfactory,

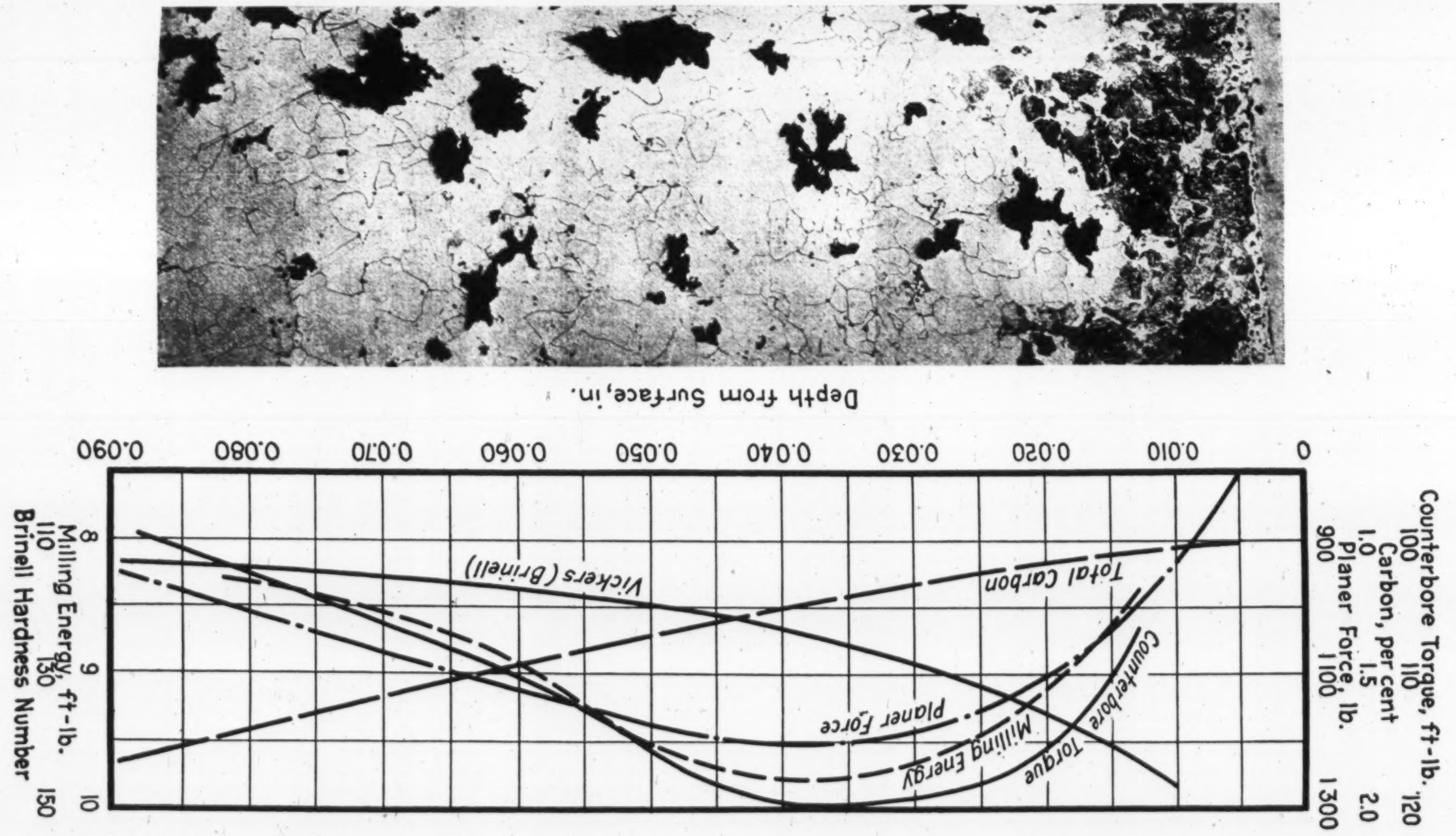


Fig. 6.—Photomicrograph of Malleable Iron Bar No. 23A (× 100), Showing the Structure from the Surface at the Top of the 1¼-in. Section. (Etched with 4 per cent Nital for 4 to 5 Seconds.) The Curves at the Right Indicate Physical and Chemical Properties at Various Depths.

so that, after the planer tests as described below were completed, another specimen was taken from the center of the $1\frac{1}{8}$-in. section of bar No. 23A immediately adjacent to the groove made by the planer tool. This specimen was first copper-plated, to insure the preservation of the extreme outer surface of the specimen, and then cast in Woods metal, after which the face corresponding with the side of the groove made by the planer tool was polished, etched, and photomicrographed. This photomicrograph is shown in Fig. 6. The outside surface, or the small layer at the top of Fig. 6, appears to consist of a layer of almost pure ferrite about 0.001 in. thick. A pearlitic structure is seen from this point to a depth of 0.015 in. The structure below this point, or between depths of 0.015 and 0.090 in., appears to be rather uniform and consists of ferrite containing large segregations of graphite. This same structure continues to extend well into the center of the bar.

TABLE II.—HARDNESS AND TENSILE PROPERTIES OF MALLEABLE IRON TENSION TEST BARS.

| Bar | Hardness Tests | | Tension Tests | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Rockwell "B" Scale, Average | Brinell Average | Diameter | Area | Yield Load, lb. | Yield Point, lb. per sq. in. | Breaking Load, lb. | Ultimate Strength, lb. per sq. in. | Elongation in 2 in., per cent |
| No. 21........... | 71.6 | 140 | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| No. 22A.......... | 62.7 | 121 | 0.630 | 0.312 | 10 610 | 34 100 | 17 590 | 56 340 | 18 |
| No. 22B.......... | 63.1 | 124 | 0.635 | 0.317 | 11 610 | 36 600 | 18 030 | 56 880 | 14.8 |
| No. 23A.......... | 62.4 | 131 | 0.645 | 0.327 | 11 900 | 36 200 | 18 380 | 56 210 | 23.4 |
| No. 23B.......... | 60.8 | 131 | 0.640 | 0.322 | 11 420 | 35 500 | 18 160 | 56 400 | 24.3 |

The hardness and tensile properties of the malleable iron are shown in Table II. Most of the values were determined from the tension test bars. No tension test bar was furnished with machineability bar No. 21, while two were furnished with each of the machineability bars Nos. 22 and 23, and are designated as A and B respectively. These tension test bars were from the same heat and anneal as the corresponding machineability bar. The hardness tests on bar No. 21, Rockwell and Brinell, were determined on a ground surface on the machineability bar. This ground surface was approximately $\frac{1}{16}$ in. below the outside surface. Hardness values for the other bars were determined on surfaces ground on the cylindrical side at the end of each of the tension test bars which gave a surface $\frac{3}{8}$ in. wide and approximately $\frac{1}{16}$ in. below the original outside surface at the center. The Brinell reading was taken $\frac{5}{8}$ in. from the end of the bar, while four Rockwell "B" scale readings were taken, two on either side of the Brinell impression. Care was taken to have these hardness values representative of the properties of the several bars.

## THE TEST METHODS

The machineability tests on the various bars consisted of measuring the force on a planer tool, the torque and thrust on a drill and counterbore, the energy required to remove a single chip by means of a pendulum type milling machine, and the time in minutes for a drill under a given load and speed to penetrate 0.100 in.

Two planer tools were used. In order to obtain a relatively high cutting force to minimize errors in reading, a planer tool of the end-cutting type 1.210 in. wide having a front-rake angle of 15 deg. was used when taking a feed of 0.005 in. per cut. This series of tests would furnish the force values for each successive cut 0.005 in. deep from the surface through the decarburized surface of the bar. A second end-cutting tool 0.500 in. wide and a 15-deg. front-rake angle was used in subsequent tests with feeds of 0.010 in. per chip. From these two tests it was concluded that the feed of 0.010 in. per cut was small enough and that the ½-in. wide tool used in previous tests on a wide variety of metals was satisfactory. In all tests covered in this paper the cutting speed was 16 ft. per minute.

In drilling, the torque, thrust, and input power were obtained when a given twist drill ¾ in. in diameter was used, operating at 153 r. p. m. with a feed of 0.012 in. per revolution. This drill was sharpened to a predetermined condition to give uniformity in successive tests and it was the same drill which had been used in previous tests on a wide variety of metals.

To overcome the disadvantage of the point angle of the drill in penetrating thin decarburized skins, the torque, thrust, and input power developed by a counterboring tool 2.495 in. in diameter having a 1-in. pilot and two horizontal cutting edges with zero degree front rake, also were determined. The speed of this counterbore was 75 r. p. m., and the feed 0.006 in. per revolution. The value of the torque, thrust, and power at any instance represents the machining properties of the metal as a function of the depth cut.

The penetration of a standardized drill ¼ in. in diameter rotating at 504 r. p. m. under a total load of 93.75 lb. was obtained and recorded in minutes to penetrate 0.100 in. Two successive cuts were run in the same hole, the first to measure the drill penetration in the so-called skin and the second to measure the drill penetration in the more uniform metal under the skin. Before the stop watch was started for the first 0.100 in., the drill was allowed to advance 0.080 in. after the drill point had come in contact with the outer surface of the metal so as to account for the length of the drill point. The drill under these conditions would be cutting full diameter for a depth below the skin of approximately 0.005 in. before the test readings were started.

The energy in foot-pounds required to remove a chip by a single-tooth milling cutter was measured. Two cutters of the end-cutting type were used as indicated, one 0.237 in. wide and the other 0.250 in. wide; both had a front-rake angle of 15 deg. The feed per chip in all cases was 0.010 in. The depth of cut for standardized cuts was 0.125 in. as this had been used in previous tests on a wide variety of metals. In some instances the depth of cut was considerably reduced so that the machineability of skin at various depths from the surface for a given chip thickness and feed could be determined.

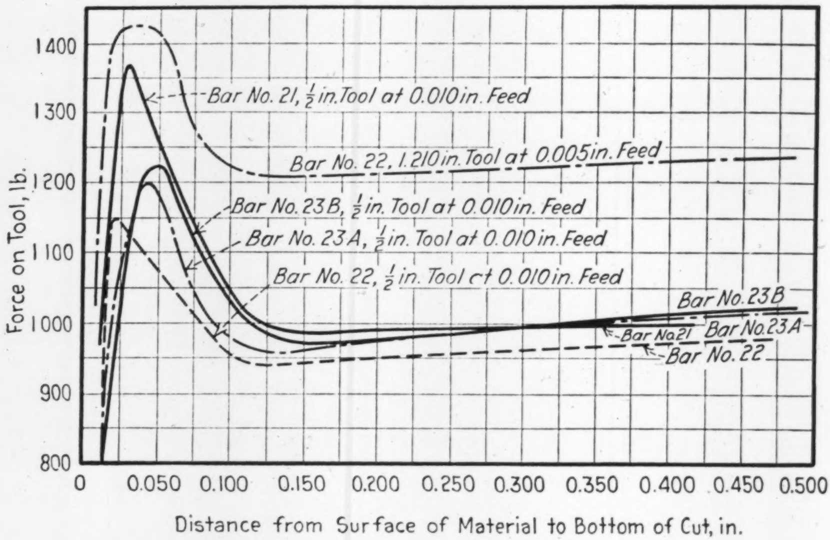


FIG. 7.—Force on Tool Required to Plane Malleable Iron at Successive Depths.

Tool ½-in. wide, 15-deg. front rake, operation at feed of 0.010 in. per cut.
Tool 1.210-in. wide, 15-deg. front rake, operation at feed of 0.005 in. per cut.
Cutting speed, 16 ft. per minute.

The various energy values given are each an average of 10 cuts. Individual cuts may vary a maximum of 10 to 12 per cent but the average of 10 consecutive cuts will agree within one to two per cent with an average similarly obtained on uniform metal.

## THE TEST DATA

*The Planer Tests:*

The first planer tests were run on bar No. 22 with the 1.210-in. wide tool having a 15-deg. front-rake but no side-rake angle. The tool was fed vertically downward for each successive cut 0.005 in. to a total depth of nearly $\frac{1}{2}$ in. The force values for each cut are shown in Fig. 7 over the distance from the surface of the material to the bottom of each cut. The cutting speed was 16 ft. per minute and the cutting was done dry. The force for the first cut was 1030 lb., for the second cut 1160 lb., for the third cut 1350 lb., for the fourth cut 1420 lb., and for the fifth cut 1420 lb., as shown plotted in Fig. 7. For the fifth cut a total depth of 0.025 in. was reached. The force for each additional cut to a depth of 0.050 remained practically constant, but after this a marked reduction was observed. The force reached a minimum value of about 1210 lb. at a total depth from the surface of 0.125 in. The force values for successive cuts remained practically constant, although a slight increase was observed until the average value of 1240 lb. at a total depth of 0.500 in. was reached.

While this force curve shown in Fig. 7 represents the machining properties as determined by the planer tool at various depths from the surface, it is not truly representative, inasmuch as the cut was taken so that the edge of the wide tool was less than $\frac{1}{16}$ in. from the edge of the bar in the $1\frac{1}{8}$-in. section. This was necessary in order to conserve the material available for subsequent tests. This particular curve is then slightly high and somewhat distorted and subsequent curves representing the force at different depths on the $\frac{1}{2}$-in. tool are believed to be more truly representative. After determining that the decarburized surface extended to approximately $\frac{1}{8}$ in. below the outer surface, it is obvious that the side of the cutting tool adjacent to the edge of the machineability bar was actually cutting decarburized material throughout the whole $\frac{1}{2}$-in. depth of cut. This would represent practically a constant cutting force on the end of the cutting tool for a length of approximately $\frac{1}{16}$ in., whereas the cutting force on the balance of the cutting-edge length varied with each cut.

Ninety-five individual readings were made in determining the curve mentioned above. By doubling the feed to 0.010 in. per cut this number of tests could be reduced to half, so that the $\frac{1}{2}$-in. tool was substituted for the 1.210-in. tool. The bottom curve of Fig. 7 represents the average of the forces for each chip in taking cuts from the surface to a total depth of $\frac{1}{2}$ in. The depth of each cut is 0.010 in., and the cutting speed is 16 ft. per minute. The cut was taken in the center of the large face of the 1⅛-in. section so that the influence of the decarburized skin on the edge of the tool was eliminated. This curve shows that the force for the first cut was 900 lb.; for the second cut 1150 lb.; for the third cut 1120 lb.; for the fourth cut 1100 lb.; for the fifth cut 1080 lb.; and for the sixth cut, at a total depth of 0.060 in., 1050 lb. The maximum value of 1150 lb., then, is reached in the second cut for the chip between the depth of 0.010 in. and 0.020 in. From this point the force reduces gradually for each successive cut to a minimum of 940 lb. at a depth of 0.125 in. This curve for bar No. 22, using the ½-in. tool, is in general similar to the curve using the 1.210-in. tool. The curve for the ½-in. tool shows more of a point at the maximum values of the force than the curve for the 1.210-in. tool, due probably to the fact that it was cutting through the skin parallel to the length of the cutting edge of the tool; whereas the wide tool, cutting near the edge of the bar, not only cuts through the skin parallel to the length of the cutting edge but also through the skin on one end of the cutting edge for the whole depth as outlined above.

Additional planer-force curves for bars Nos. 21, 23A, and 23B are also shown in Fig. 7. The ½-in. tool with a vertical feed of 0.010 in. was used in each case. Bar No. 21 has the highest maximum cutting force which reaches a value of about 1365 lb. at a depth of 0.030 in. Bars Nos. 23A and 23B appear to have machining curves quite identical, as might be expected, inasmuch as the bars were cast and annealed at the same time. For bars Nos. 23A and 23B, however, the maximum force is reached at a depth of approximately 0.050 in. It is of interest to note that the force values for all bars below a depth of ⅛ in. are practically identical.

The first and second points at the extreme left of each curve in Fig. 7, representing values for the first and second cuts taken, may be low for three reasons: the material may cut with ease, the surface may be irregular, or the surface of the castings may be warped so that the cut did not become of full size and uniform during the whole length of the first one or two strokes. There may be a spring of the tool or deflection of the work for each cut which would increase gradually with successive cuts until a certain uniformity is reached. Subsequent tests, however, appear to substantiate the results of the planer tests as reported.

Chips from the ½-in. planer tool with a feed of 0.010 in. are shown in Fig. 2 for machineability bars Nos. 21, 23A and 23B. Only enough chips are shown to indicate the tendency of formation at various depths. There is little difference between the shape of chips of corresponding depths from different bars. The chips shown correspond to the depths at which total-carbon content was determined as shown plotted in Fig. 3.

The maximum force values of the various planer tools ½ in. wide operating at a feed of 0.010 in. and a cutting speed of 16 ft. per minute when cutting dry are given in Table III. These maximum forces occur as follows:

TABLE III.—RESULTS OF METAL CUTTING TESTS.

| Bar | Thickness of Section, in. | ¾-in. Drill[a] | | | 2.495-in. Counterbore[d] | | | | | Milling Energy per Chip, ft-lb. | Drill Penetration, Minutes[g] | | Force on Planer Tool, lb.[h] | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Torque, ft-lb.[b] | Thrust, lb. | Net Input, kw.[c] | Torque | | Thrust | | Net Input, kw., Body | | First 0.100 | Second 0.100 | Maximum | Body[i] |
| | | | | | Skin | Body | Skin | Body | | | | | | |
| | | | | | | | | | | 10.7[e] | | | | |
| No. 21. | 1⅛ | 17.2 | 590 | 0.48 | ... | 91 | 505 | 390 | 1.07 | 7.95[f] | 0.122 | 0.129 | 1365 | 985 |
| | ¾ | 16.75 | 600 | 0.47 | 109 | 89 | 500 | 380 | 1.19 | 8.05[f] | 0.119 | 0.125 | | |
| | ¼ | .... | ... | .... | 112 | 67 | 400 | 300 | 0.97 | 7.73[f] | 0.120 | .... | | |
| | | | | | | | | | | 10.05[e] | | | | |
| No. 22. | 1⅛ | 16.7 | 590 | 0.46 | 121 | 90 | 535 | 370 | 1.17 | 8.05[f] | 0.120 | 0.125 | 1150 | 940 |
| | ¾ | 16.5 | 580 | 0.44 | 112 | 86 | 500 | 350 | 1.12 | 7.95[f] | 0.119 | 0.123 | | |
| | ¼ | .... | ... | .... | 109 | 66 | 400 | 250 | 0.98 | 7.65[f] | 0.118 | .... | | |
| No. 23A | 1⅛ | 17.4 | 590 | 0.47 | 119 | 90 | 580 | 400 | 1.17 | 10.8[e] | 0.124 | 0.130 | 1200 | 960 |
| | ¾ | 17.3 | 590 | 0.47 | 116 | 86 | 500 | 370 | 1.15 | .... | 0.118 | 0.124 | | |
| | ¼ | .... | ... | .... | 117 | 67 | 430 | 370 | 0.94 | .... | 0.117 | .... | | |
| No. 23B | 1⅛ | 17.1 | 584 | 0.48 | 121 | 92 | 500 | 410 | 1.15 | 10.80[e] | 0.124 | 0.130 | 1225 | 970 |
| | ¾ | 17 | 580 | 0.47 | 109 | 86 | 520 | 320 | 1.15 | .... | 0.113 | 0.120 | | |
| | ¼ | .... | ... | .... | 118 | 67 | 500 | 300 | 0.94 | .... | 0.118 | .... | | |

[a] Rotating at 153 r.p.m. with a feed of 0.012 in., cutting dry.

[b] The values for torque given for the 1⅛-in. thick section are an average of four tests in different parts of the thick section, see Fig. 1(*b*).

[c] The net input in kw. is equal to the gross minus the tare as determined from the wattmeter chart.

[d] The counterboring tool was rotated at 74 r.p.m. at a feed of 0.006 in. per revolution.

[e] This value is for a cutting tool 0.250 in. wide having 15-deg. front rake, and represents an average value of ten successive cuts.

[f] The milling cutter for these tests was 0.237 in. wide. Each value is an average of ten successive cuts.

[g] The ¼-in. diameter drill used rotated at 500 r.p.m. under a feed load of 93.75 lb.

[h] The tool was end-cutting ½-in. wide operating at a feed of 0.010 in.

[i] Read from the curves of Fig. 7 at a depth of 0.150 in. from the surface.

For bar No. 21, 1365 lb. at 0.030-in. depth
For bar No. 22, 1150 lb. at 0.020-in. depth
For bar No. 23A, 1200 lb. at 0.040-in. depth
For bar No. 23B, 1225 lb. at 0.050-in. depth

The planer-tool force at a depth of 0.150 in. is shown in Table III for the respective bars opposite the maximum force.

*Drilling Tests for Torque, Thrust, and Power with a ¾-in. Drill:*

The torque, thrust, and net power input for the ¾-in. drill for machineability bars Nos. 21, 22, 23A, and 23B are shown in Table III. Six holes were drilled, Nos. 1 to 6 inclusive, as shown located in Fig. 1 (*b*). The values for cuts Nos. 1, 2, 5, and 6 were averaged for representative figures for the 1⅛-in. section. Cuts Nos. 1 and 2 were perpendicular to the face of the casting and centrally located and cuts Nos. 5 and 6 were perpendicular to the edge through the center. Cut No. 3 was taken through the center of the ¾-in. section and cut No. 4 was taken through the center of the ¼-in. section. The values for cut No. 4 appear to be very consistent with those of the other cuts, but because of the thinness of the section the point of the drill was nearly through the bar before a full diameter was cut, so that full values of torque, thrust, and power were obtained only for a very short duration. They were left out of consideration in this paper. The torque values for this thin section seem to agree very well with the corresponding values for the heavier sections. The thrust, however, is about 10 per cent less, due either to the ease of machining the metal or to the fact that before the drill point pierced the lower side the metal bulged, thereby giving relatively low values of thrust. The values representing torque, thrust, and power in Table III for the ¾-in. thick section is the result of only a single cut. It is not safe, therefore, to rely too much upon the relation between the values of the 1⅛ and the ¾-in. sections. In tests of this nature, maximum differences of 4 or 5 per cent may occur among individual tests. The average of four or five tests, however, will not differ by more than 1 per cent from another similar average if the metal is uniform.

In these drilling tests the influence of the decarburized skin on the torque or thrust was not noticeable. The standard 118-deg. point angle of the drill was such that by the time the drill was taking a full ¾-in. cut, the point itself was actually through the decarburized skin.

*The 2.495-in. Counterboring Test Results:*

After drilling the ¾-in. holes Nos. 1, 2, 3, and 4 in the machineability bars as shown in Fig. 1 (*b*), they were enlarged to 1 in. in diameter to take the pilot of an available counterboring tool. The cutting edges of the counterboring tool were in a straight horizontal line. The overall length was 2.495 in. The torque, thrust, and resulting power as recorded give a true indication of the machineability of the metal at all depths for this tool. Table III shows the values of torque and thrust for the counterboring tool when penetrating the skin and when cutting the interior or body of the castings. These values are shown for each of the three thicknesses of the castings. The location of these counterboring cuts is shown in Fig. 1 (*c*). The values given for the 1⅛-in. section are the average of two tests in that section. It was noticed that for bars Nos. 21 and 22, the skin and body torque values were about 3 per cent higher for cut 1 at the end of the bar than for cut 2. The values for the ¾ and ¼-in. sections represent the results for one single test on each bar. The torque values for the skin, that is, the peak of the curves, are shown to be consistently higher in all tests than for the average torque values after penetrating the ⅛-in. thick skin. The skin torque for bar No. 21 was not obtained. The maximum skin torque for the 1⅛-in. sections is 121 ft-lb. for bar No. 22, 119 ft-lb. for bar No. 23A, and 121 ft-lb. for bar No. 23B. The maximum values of the skin torque for the ¾-in. sections is 109 for bar No. 21, 112 for bar No. 22, 116 for bar No. 23A, and 109 for bar No. 23B. The values for bars Nos. 21 and 23B appear to be low and out of order, whereas the value for bar No. 22 is lower than that for bar No. 23A but conforms favorably with the other machineability data. The maximum skin-torque values for the thinnest sections appear to be lower for all bars than the corresponding values for the other sections, indicating, perhaps, that the decarburized skins on the thin sections machine easier than the skin on the thicker sections.

The average body torque values for the 1⅛-in. sections appear to be approximately the same for all bars. The body torque for the ¾-in. section of bar No. 21 is higher than these values for the other three bars, being 89 ft-lb. for bar No. 21 as compared to 86 for the others. This conforms with the results of the planer-force curves shown in Fig. 7 in which bar No. 21 has the highest value. There appears to be little consistent difference between the skin thrust values on the counterboring tool for the 1⅛-in. and the ¾-in. sections. For the ¼-in. section, however, on bars Nos. 21, 22, and 23A, abnormally low values are shown. These low values may be due to the fact that the material bulged on the underside because of the thrust and thereby relieved the pressure. A more adequate support of the specimen when drilling with the ¾-in. drill and counterboring tool is necessary.

The fact that the body torque for the ¼-in. section is lower than the torque value for the thicker sections may also be due to the effect of bulging caused by the thrust on the counterboring tool. It is of interest to note, however, that for bar No. 23B the body torque value for the ¼-in. section is 67 ft-lb. and equal to the corresponding torque for the other bars, while the thrust value of 500 lb. appears to be quite normal.

*Milling Cutter Test Results:*

Two types of tests were conducted with the single-tooth milling cutter dynamometer: one, to determine the machining characteristics of a bar so as to compare it with similar characteristics of another bar; the other to determine the machining characteristics of the decarburized skin for the sake of comparing it with similar data obtained by other metal-cutting processes.

In Table III are shown milling energy values in foot-pounds required to remove a given chip with a single-tooth cutter 0.250 in. wide when taking a depth of cut from the outside surface of the casting of 0.125 in., and a feed per chip of 0.010 in. These values for bars Nos. 23A and 23B are practically identical, being 10.8 and 10.78 ft-lb., respectively. Bar No. 21 has a slightly lower value of 10.7 ft-lb., while bar No. 22 has the lowest value of 10.05 ft-lb. per chip. It was found that these values will vary slightly at different locations on the same thickness of the machineability bar. The milling energy value for a depth of $\frac{1}{8}$ in. actually represents an average value for a metal which varies considerably in properties in this depth.

In order to compare the machining properties of the skin on the sections of different thickness, a newly ground milling cutter 0.237 in. wide was used in taking a similar cut from the face of the $1\frac{1}{8}$, $\frac{3}{4}$ and $\frac{1}{4}$-in. sections. A feed of 0.010 in. per chip was used. For a depth of cut of 0.125 in. the energy in foot-pounds per chip on the $1\frac{1}{8}$, $\frac{3}{4}$, and $\frac{1}{4}$-in. sections of bar No. 21 were found to be 7.95, 8.05, and 7.73, respectively, as shown in Table III. For bar No. 22, corresponding values were found to be 8.05, 7.95, and 7.65, respectively. Similarly, on bar No. 21 in taking a depth of cut of 0.025 in., with a feed of 0.010 in., values of 3.0, 2.9, and 2.7 ft-lb. per chip were determined for the $1\frac{1}{8}$, $\frac{3}{4}$, and $\frac{1}{4}$-in. sections, respectively. The milling energy required to take the 0.125 in. chip from the $\frac{3}{4}$-in. section of bar No. 21 is slightly greater than the energy required to remove the chip from the $1\frac{1}{8}$-in. section. Otherwise, all of the data obtained in these tests show that slightly less energy is required to remove the skin by milling from the $\frac{3}{4}$-in. section than from the $1\frac{1}{8}$-in. section. In all cases the energy required to remove the chip from the $\frac{1}{4}$-in. section is least of all.

The results of experiments with the single-tooth milling cutter in removing metal from machineability bar No. 23A, taking the first chip from the surface to a depth of 0.025 in. with a feed of 0.010 in., are shown in Fig. 8. The energy in foot-pounds per chip was determined for the first chip from the surface, after which a second cut was taken in the bottom of the first cut to an additional depth of 0.025 in. and the energy determined. These energy values for several successive cuts are shown plotted in Fig. 8 against the depth to the center of the cut from the surface. The energy value for removing a chip to a depth of 0.025 in. really represents an average value for the metal from the surface to a depth of 0.025 in. The second point

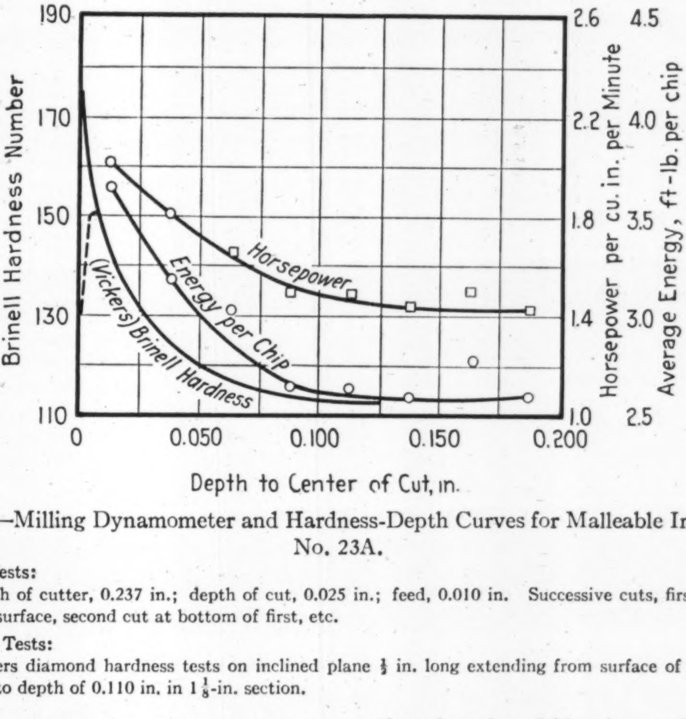


FIG. 8.—Milling Dynamometer and Hardness-Depth Curves for Malleable Iron Bar No. 23A.

Milling Tests:
Width of cutter, 0.237 in.; depth of cut, 0.025 in.; feed, 0.010 in. Successive cuts, first cut at surface, second cut at bottom of first, etc.

Hardness Tests:
Vickers diamond hardness tests on inclined plane ½ in. long extending from surface of test bar to depth of 0.110 in. in 1⅛-in. section.

on the curve represents an average value for the chip removed from between 0.025 in. depth and 0.050 in. depth. This curve is similar to the planer-force curve for bar No. 23A shown in Fig. 7, in that the energy decreases for the same thickness at various distances below the surface until a depth of approximately 0.125 in. below the surface is reached. The energy reduced to horsepower per cubic inch per minute of metal removed is also shown in Fig. 8. The energy curve shows the first point to have the highest value of 3.75 ft-lb. This would indicate that the material for the first 0.025-in. depth requires more energy to remove it than does the material of the second 0.025 in. depth.

It was thought desirable to continue the study of metal removal to determine the energy in foot-pounds required to remove a chip having a depth of cut less than 0.025 in. at increasing depths below the surface to see if the results obtained would be similar to those for the 0.025-in. depth and to see if the influence of the ferrite skin could be determined more accurately. It was felt that there should be a greater correlation between these energy curves for the milling cutter for small depths of cut and corresponding cuts of the planer tool.

Again, using bar No. 23A with a cutter 0.250 in. wide, operating at a feed per chip of 0.010 in., various cuts were taken, the first cut at the surface and the second cut in the bottom of the first cut, etc., with different depths of cut. These values are shown in Fig. 9 in which four curves are shown. The highest curve represents the milling energy values required to remove a chip 0.025 in. deep. It is seen, therefore, that 2.8 ft-lb. are required to remove a chip extending from the surface to a total depth of 0.025 in. and 2.75 ft-lb. are required to remove the chip of 0.025 in. depth, extending from the bottom of the first cut to a total depth of 0.050 in. The second highest curve in Fig. 9 shows similar energy values for removing chips at different depths, each cut of which is 0.020 in. in depth. The third highest curve shows milling energy values required to remove chips of 0.010 in. in depth at different distances below the surface. The lowest curve shows various energy values for removing chips of 0.005 in.

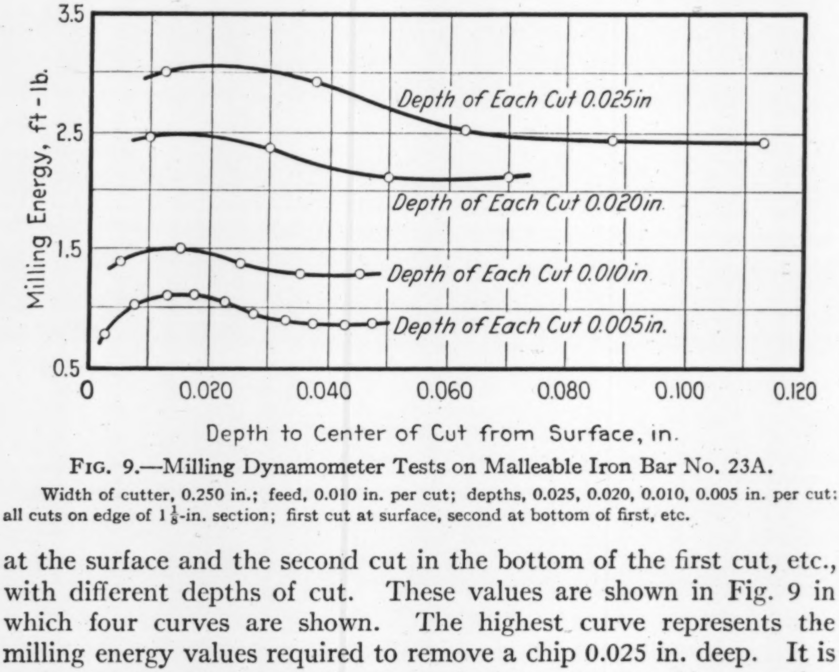


FIG. 9.—Milling Dynamometer Tests on Malleable Iron Bar No. 23A.

Width of cutter, 0.250 in.; feed, 0.010 in. per cut; depths, 0.025, 0.020, 0.010, 0.005 in. per cut: all cuts on edge of 1 1/8-in. section; first cut at surface, second at bottom of first, etc.

depth at successive depths below the surface. The lowest curve for a depth of cut of 0.005 in. was used to show that the ferrite surface is removed with less energy than are chips of the same thickness on the interior.

These results show the general influence of the decarburized surface on the milling energy factor, but inasmuch as they were taken on the edge of the 1⅛-in. section, they will not correspond exactly to values taken on the face of the 1⅛-in. section. The tests had to be run on the edge of the section because of insufficient space left on the face.

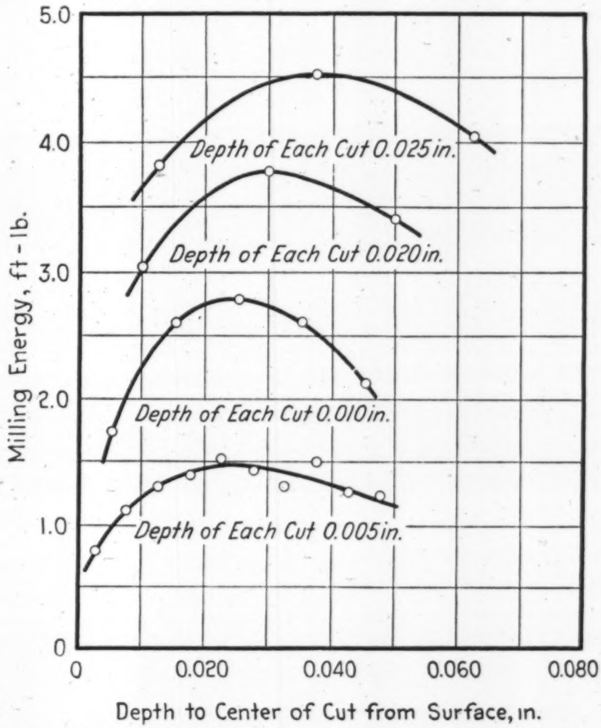


FIG. 10.—Milling Dynamometer Tests on Malleable Iron Bar No. 21.

Width of cutter, 0.250 in.; feed, 0.010 in. per cut; depths, 0.025, 0.010, 0.005 in. per cut; all cuts on face of 1½-in. section; first cut at surface, second at bottom of first, etc.

As a further check on this study, similar milling cutter tests were run on the face of the 1⅛-in. section of bar No. 21. Four curves representing the energy required to remove chips of 0.005, 0.010, 0.020, and 0.025 in. deep, respectively, at various depths from the surface are shown in Fig. 10. Here again the width of the cutter was

0.250 in. and a 15-deg. front rake was provided. The feed per chip was 0.010 in. These curves all conform to the general shape of the planer-force curve for bar No. 21 as shown in Fig. 7. The curves represented in Fig. 10 show much lower values of milling energy for the first cut from the surface for each depth, due, it is believed, to the fact that the cuts were taken in the center of the face of the thickest section of the machineability bar.

*Results of $\frac{1}{4}$-in. Drill Penetration Tests:*

The results of the drill penetration tests for the four machineability bars are given in Table III. Each value is expressed in minutes for the drill to penetrate 0.100 in. after the drill point has actually penetrated the surface a distance of 0.080 in. It was determined that the drill point had to be buried into the metal 0.075 in. before the lips of the drill took a full diameter cut. An allowance of 0.005 in. was made for irregular surface, etc., before the actual timing was started. It is seen in Table III that the time for the drill to penetrate the face of the $1\frac{1}{8}$-in. section of bar No. 21 the first 0.100 in. after the drill point has penetrated 0.080 in. is 0.122 minutes. The time for the drill to penetrate the second 0.100 in. was found to be 0.129 minutes. These values represent averages of four to eight individual tests. In many instances these averages have been checked to within 1 or 2 per cent. In determining the time for the $\frac{1}{4}$-in. drill to penetrate the $\frac{3}{4}$-in. section of bar No. 21 for the first 0.100 in., an average value of 0.119 minutes is shown, while an average time for the second 0.100 in. was found to be 0.125 minutes. The average time to penetrate the $\frac{1}{4}$-in. section of bar No. 21 is shown to be 0.120 minutes for the first 0.100 in. Values for the second 0.100 in. were not determined, inasmuch as the drill point would have passed the central part of the $\frac{1}{4}$-in. thick section. Similar values for the time of penetration of the first and second 0.100 in. are shown for bars Nos. 22, 23A, and 23B in Table III. It is interesting to note that all observations point to the fact that more time is required to penetrate the second 0.100 in. than is required to penetrate the first, in spite of the fact that the greatest cutting forces occur in the first 0.100 in. of depth. It is also clear from the data presented that the first 0.100 in. penetration of the $\frac{3}{4}$-in. section is made in less time than that required for the $1\frac{1}{8}$-in. section. For bar No. 21, 0.122 minutes are required to penetrate the first 0.100 in. in the $1\frac{1}{8}$-in. section, while 0.119 and 0.120 minutes are required for the first 0.100 in. penetration in the $\frac{3}{4}$-in. and $\frac{1}{4}$-in. thickness, respectively. For bar No. 22, the time for penetrating the first 0.100 in. is 0.120 minutes for the $1\frac{1}{8}$-in.

section, 0.119 minutes for the $\frac{3}{4}$-in. section, and 0.118 minutes for the $\frac{1}{4}$-in. section. Similar values are obtained for bars Nos. 23A and 23B. The least time element for the first and second 0.100 in. penetrations is for the $\frac{3}{4}$-in. section of bar No. 23B, whereas bar No. 22 appears to be machined easiest by the other cutting processes.

*Vickers Diamond Hardness Test Results:*

It was thought desirable to determine the hardness of a specimen of malleable iron at various depths below the surface. A specimen $\frac{1}{2}$ in. wide, $\frac{1}{2}$ in. deep, and $1\frac{1}{2}$ in. long was removed from bar No. 23A from a portion immediately adjacent to the groove made by the planer tool in securing the planer-force curve of Fig. 7, and also immediately adjacent to the specimen removed for the preparation of the photomicrograph shown in Fig. 6. This should permit a correlation between the results of hardness tests at different depths with the planer-tool force at the same depths, as well as the structure shown in the photomicrograph.

A Vickers hardness testing machine, employing a diamond pyramid point, was used which gave hardness numbers equivalent to the Brinell machine. The test specimen was bevelled on the top to give a smooth plane surface. The tests were started from the outside surface near one end of the specimen, dropping to a depth of 0.125 in. at the other end. Conflicting results were obtained on the surface immediately beneath the skin. A Brinell hardness curve shown in Fig. 8 represents an average of the results obtained. The hardness, expressed in Brinell numbers, is nearly 180 at the surface, but drops rapidly to an average of 113 at 0.100 in. below the surface. There was little further reduction in hardness at greater depths. Several sets of readings were taken, some of which showed a maximum Brinell hardness of approximately 150 at a distance of 0.010 in. below the surface. Between this point and the surface, some lower values were obtained. The Brinell hardness curve shown in Fig. 8 indicates both tendencies of high and low values up to a depth of 0.010 in. Data which seemed to be conclusive could not be obtained due, presumably, to the fact that the actual point of contact between the material and the diamond point was so small that the results were materially influenced by the non-uniformity of the structure.

## DISCUSSION OF EXPERIMENTAL DATA

The experimental data obtained from the various methods of tests have been outlined above in quite some detail. It may be well, however, to point out the coordinating or conflicting results.

There appears to be complete coordination between the total-carbon content of the decarburized skin, as shown at different depths for each bar in Fig. 3. and the force on the planer tool as a function of the depth from the outside surface of the bar. It appears that the greater the effect of decarburization, that is, the lower the carbon content near the surface, the greater is the cutting force involved, as indicated for bar No. 21 in Figs. 3 and 7. Similarly, bar No. 22, having the highest carbon content near the surface, has the lowest machining force as determined on a planer tool as shown in Fig. 7. This is further borne out by bar No. 23B which has a low carbon content near the surface, as shown in Fig. 3, being nearly as low as that of bar No. 21, while its planer-force values are nearly as high as those for bar No. 21, as shown in Fig. 7.

In Fig. 3 it is seen that the total-carbon content of bar No. 21 is higher than for all other bars for depths at or below 0.100 in. From Fig. 7 it is seen that the machineability, as represented by the force on the planer tool at this depth, also is highest for bar No. 21, indicating perhaps that high carbon content indicates high force in machining. In Fig. 3 it is shown that below a depth of 0.120 in., the total-carbon content of bar No. 22 is lower than that for the other three bars. In Fig. 7 it is also seen that the force on the planer tool in this range of depth in bar No. 22 is lowest of all of the four bars.

The chips removed by the planer tool, Fig. 2, show by their shape and consistency a direct relation to the carbon content of the decarburized skin, as shown in Fig. 3. The photomicrographs do not show complete coordination between the structure of the metal and the total-carbon content of the decarburized skin as determined by analysis, nor the force on the planer tool as determined experimentally. It may be that the photomicrographs, covering such a small area of the structure as they do, were selected at just the wrong points. The structure indicated by the photomicrograph of malleable iron bar No. 23A, shown in Fig. 6, was determined on the face of a specimen located immediately adjacent to the point at which the planer-tool force and Vickers hardness numbers were determined and yet planer-force and hardness values, which agree in themselves for different depths from the surface, cannot be explained from the structure shown in the photomicrograph.

There appears to be a satisfactory correlation between the results of the planer-force, Vickers (Brinell) hardness, and milling-energy values as expressed in terms of depth of decarburized case. The milling-energy values show the cutting characteristics of the decarburized case when the depth of successive cuts is smallest, as shown in Figs. 9 and 10.

Figure 6 has been prepared so as to show the various machineability factors, the Vickers hardness, and total-carbon content as a function of the structure of malleable iron bar No. 23A from the surface inward.

The total-carbon curve represents the percentage of total carbon determined from chips removed by the planer tool at various depths, each chip being 0.010 in. in thickness. A point on the curve, therefore, represents the total-carbon content of that section of the photomicrograph 0.005 in. to the right and left. A point on the planer-force curve also represents the force on a planer chip 0.010 in. deep, which is 0.005 in. above and below the point indicated. In other words, the values on the total-carbon curve, the planer-force curve, and the milling-energy curve are plotted over the center of the depth of the cut. The counterbore torque is more continuous and, therefore, the points on this curve correspond with the exact depth from the surface. This also holds true for the Vickers hardness numbers which correspond to the Brinell scale.

The close relation between the planer-force curve, the milling-energy curve, and the counterbore-torque curve is interesting, inasmuch as they are all low at the surface of the metal, all rising to a maximum at a depth of approximately 0.035 to 0.040 in. from the surface, and again falling off to a low value at the greatest depth. These values continue to drop until a total depth from the surface of 0.100 to 0.125 in. is reached.

Obviously, the machineability factors are not wholly dependent upon the structure of the metal as indicated by the total-carbon content, inasmuch as the total carbon is lowest at the surface and highest at the greatest depth of 0.090 in. shown. These machining factors, therefore, must be a function of the structure. It is interesting to note that the maximum machineability values do not occur in the ferrite skin or even in the pearlitic structure just below the skin, but rather in a comparatively uniform structure of ferrite and graphite at some distance below the pearlitic structure. No explanation is made of this relation at this time.

Just why the machining values are lowest near the surface is not clear. It may be that the pearlitic structure actually machines with greater ease than the ferrite structure. It seems plausible, however, that these low values may be due to the fact that the depth of the pearlitic structure corresponds with the depth of the planer-tool cut so that there is a peeling action; that is, the pearlitic layer is peeled off rather than cut off. This explanation does not seem to hold, however, in the case of the counterbore-torque curve or the milling-energy curve. In the case of the counterbore torque, the values obtained are continuous, the actual thickness of chip being 0.003 in., which is one-half the feed of the counterboring tool. The depth of cut of the milling tool was 0.025 in., which actually overlaps the margin of the pearlitic structure. Obviously, additional information is needed on this point.

It is interesting also to note that the hardness as determined on the Vickers machine is practically the reverse of the total-carbon curve, being a maximum at a depth of 0.010 in. below the surface and falling off gradually to a minimum at a depth of 0.090 in. below the surface. No satisfactory explanation of this relation is indicated at this time.

## CONCLUSIONS

It must be borne in mind that the results of this paper have been based on tests conducted on but four small machineability bars. Malleable cast iron is shown to vary considerably in its structure and properties throughout the casting. Many of the experimental data presented above are the result of a single test or, at best, the average of very few tests, so that proper allowance must be made in drawing final conclusions. It was found, for instance, that in drilling with the $\frac{3}{4}$-in. drill and counterboring with the 2.495-in. counterbore at two places centrally located in the $1\frac{1}{8}$-in. sections that the results obtained in drilling or counterboring the hole near the end of the bar, cut 1 in Figs. 1 (*b*) and 1 (*c*), were often higher than similar values obtained from cut 2. The data presented were averages of the results of the two tests for the $1\frac{1}{8}$-in. section. While the total-carbon content of the annealed bars, as shown in Table I, appears to be practically alike for all bars (slightly high for bar No. 21), this does not seem to be a basis for changing machining characteristics of the individual bars because of the lack of equality of the other elements. In short, the author is conscious of a lack of experimental data in drawing the following conclusions:

1. From the torque of the $\frac{3}{4}$-in. drill it appears that the interior of the $\frac{3}{4}$-in. section is cut easier in the case of every bar than is the material in the $1\frac{1}{8}$-in. section. The values of thrust for these two sections substantiate in a general way the conclusions drawn from the torque values, although the evidence is less conclusive.

2. The maximum torque developed by the 2.495-in. counterbore in the decarburized skin appears to be greatest in the skin on the thickest section and least in the skin on the thinnest section, indicating that, so far as the skin is concerned, that on the thinnest section

machines easiest. This conclusion is substantiated by the values of thrust.

The torque developed in the body of the casting by the 2.495-in. counterbore indicates in the case of all bars that the torque is greatest in the section of greatest thickness and least in the section of least thickness. The thrust also confirms the results indicated by the torque.

3. The energy absorbed by a tooth of a milling cutter seems to be greatest when removing the skin on the section of greatest thickness and least when removing the skin on the section of least thickness. This holds for various depths of cut from the surface when the feed is constant.

4. The time in minutes for a $\frac{1}{4}$-in. drill working under constant feed load to penetrate the first 0.100 in. of the decarburized skin on the surface of the casting appears to be greatest when penetrating the skin of the section of greatest thickness and least when penetrating the skin of the section of least thickness. It appears also that the time required for the drill to penetrate the second 0.100 in., that is, after passing through the decarburized skin, is greater when cutting in the section of greater thickness and smaller when cutting in the section of smaller thickness. This conclusion is based on a comparison of the $1\frac{1}{8}$-in. section with the $\frac{3}{4}$-in. section. All drill penetration data show that the time required for the drill to penetrate the first 0.100 in., or the decarburized skin, of the casting is less than the time required for the drill to penetrate the second 0.100 in. This condition obtains for both the $1\frac{1}{8}$-in. sections and the $\frac{3}{4}$-in. sections.

5. The force on a $\frac{1}{2}$-in. wide end-cutting type of planer tool when fed for successive cuts vertically into the casting at the rate of 0.010 in. per cent is a representative factor of the machineability of the casting at different depths below the surface. For all castings studied, the force on the planer tool per cut is low for the first cut; for successive cuts rises to a maximum at a depth varying from 0.020 to 0.050 in. below the surface, after which it falls off again until a minimum value is reached at a depth of approximately 0.100 to 0.125 in. below the surface.

6. There appears to be a satisfactory correlation between the various methods of determining machineability for the skin and body of the sections of the castings of different thickness. The results of the counterboring tests, the milling-energy tests, and drill-penetration tests on the decarburized skin all indicate satisfactorily that the decarburized skin on the $1\frac{1}{8}$-in. section machines with greater difficulty than does the skin on the thinner sections. The skin on the thinnest

section machines easiest of all. It appears that the body or interior part of the casting machines with greater difficulty in the sections of greater thickness, as indicated for the $1\frac{1}{8}$ and $\frac{3}{4}$-in. sections of all bars as determined by the drill and counterbore torque. The counterbore torque which permits the testing of the thinnest or $\frac{1}{4}$-in. section shows that the body of the $\frac{1}{4}$-in. section machines easiest of all sections.

7. It appears that the hardness of the skin at various distances below the surface of the casting indicates quite favorably the results of metal cutting tests at these depths.

8. A complete correlation between photomicrostructure at various depths below the surface and the various machining properties does not seem to be satisfactory.

9. It appears that the decarburized skin having the lowest total-carbon content machines with greater difficulty than the skin with higher carbon content.

10. It also appears that the interior of the casting of higher total-carbon content machines with greater difficulty than the interior of the metal of lower carbon content. This conclusion is based on the results shown graphically in Figs. 3 and 7 in which the differences in total carbon are very small, as are the differences in machining properties. These differences in machining properties may be due to other factors about which too little is known to make them evident.

The maximum total-carbon range between bars Nos. 21 and 22 at a depth of 0.140 in. is 8 per cent; the maximum difference in machining properties of the body of these two bars is approximately as follows:

| | |
|---|---|
| For the force on the planer tool | 4.6 per cent |
| For the counterboring torque | 2 per cent |
| For the $\frac{3}{4}$-in. drill torque | 3 per cent |
| For the drill penetration | 3 per cent |

11. It is interesting to note that the total-carbon content of the skin, as shown in Fig. 3, does not correspond to the total-carbon content of the body of the casting, so that it cannot be assumed that low carbon content of the casting as a whole will result in low carbon content of the skin with consequent higher machining force. It appears that the carbon content of the skin is largely a function of the annealing process and not of the carbon content as cast. Inasmuch as it has been shown that the decarburized skin of all castings machines with greater difficulty than the interior, it is believed that the machining qualities of malleable iron castings would be improved materially by adopting such methods of annealing as to reduce or prevent surface decarburization.

*Acknowledgments:*

This work has been done as a project of the Department of Engineering Research of the University of Michigan. It was sponsored by the Malleable Iron Research Institute through its Research Committee. The author is indebted to the members of the Research Committee, to the staff of the Institute, and to the foundries who submitted the specimens, for their assistance and suggestions. The author also wishes to express his appreciation to Mr. R. Paul Harrington and Mr. Charles E. Kraus, who conducted the tests and assisted in the compilation of the report.

## APPENDIX I

To indicate the relative machineability of malleable iron and a number of other metals, Table IV is given. Machining properties of malleable iron bar No. 21 were selected as representing malleable iron. It is hoped that similar information may later be added representing machineability of malleable iron bars of high, intermediate, and low carbon content. The present table is offered, however, for what it may be worth, inasmuch as the malleable iron bar No. 21 may not be truly representative of a given type of malleable iron casting.

Analyses, heat treatments, and photomicrographs are given for the respective metals in the papers from which the machineability data are taken. Bars Nos. 11-1 to 32, inclusive, and Nos. 50-A to DM, inclusive, were taken from Tables V and VI of the author's paper on "Methods of Tests for Determining the Machineability of Metals in General, with Results."[1] Bars Nos. 412 to 426, inclusive, were taken from Tables III and V of the author's paper on "Machining Properties of Some Cold-drawn Steels."[2]

The various machineability values for the materials other than malleable iron represent the average of a number of tests at various locations on the bars and at different depths beneath the surface. Due to the non-uniformity of the malleable iron bar, in which a decarburized skin is present, machining characteristics of the decarburized skin as well as those for the interior of the casting are given for those tests which deal with the 0.100 to 0.125-in. depth. If the malleable iron castings were to be machined by taking a cut reaching to a depth of 0.100 to 0.125 in., the higher values should be used. If the interior of the casting is to be machined, then the lower values are to be used.

The data presented for the various metals have been accumulated over a long period of time. The tools and methods of tests have been so standardized, however, that most of these data have been checked at various intervals and, therefore, are believed to be comparative.

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[1] *Transactions*, Am. Soc. Steel Treating, Vol. XVI, No. 6, November, 1929.

[2] Advance paper No. 9 for the annual meeting of the American Society of Mechanical Engineers, December 1 to 5, 1930.

TABLE IV.—VARIOUS VALUES REPRESENTING MACHINEABILITY OF MALLEABLE IRON BAR NO. 21 LISTED FOR COMPARISON WITH SIMILAR VALUES FOR A VARIETY OF METALS.

| Bar | Material | Force on Planer Tool | | | $\frac{3}{4}$-in. Drill | | | | Milling 0.125-in. Depth | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | "F" Total Force, lb. | "U" Unit Force, lb. per sq. in. | Horse Power per cu. in. per minute | "T" Torque, ft-lb. | "Th" Thrust, lb. | Net Horse Power per cu. in. per minute | $\frac{1}{4}$-in. Drill Penetration, minutes per 0.10 in. | ft-lb. per Chip | Horse Power per cu. in. per minute |
| No. 21.... | Malleable Iron............ | 1195[a] | 239 000[a] | 0.603[a] | 16.9 | 565 | 0.610 | 0.122[c] | 10.70[e] | 1.03[e] |
| | | 995[b] | 199 000[b] | 0.502[b] | .... | .... | .... | 0.129[d] | 8.87[f] | 0.86[f] |
| No. 11-1.. | Cast Iron, 1-in. square[g]...... | 1022 | 204 400 | 0.516 | 17.0 | 600 | 0.614 | 0.107 | 7.14 | 0.68 |
| No. 28.... | S.A.E. No. 1020, large forging | 1724 | 344 800 | 0.870 | 32.5 | 1000 | 1.170 | 0.240 | 12.88 | 1.25 |
| No. 29.... | S.A.E. No. 2345, large forging | 1830 | 366 000 | 0.923 | 30.5 | 1250 | 1.10 | 0.264 | 12.20 | 1.18 |
| No. 30.... | S.A.E. No. 2320, large forging | 1749 | 349 800 | 0.884 | 28.5 | 1100 | 1.03 | 0.290 | 16.21 | 1.57 |
| No. 31.... | S.A.E. No. 3120, large forging | 1702 | 340 400 | 0.859 | 30.0 | 1100 | 1.081 | 0.261 | 11.88 | 1.15 |
| No. 32.... | Tool Steel, 1.03 per cent carbon | 2394 | 478 800 | 1.209 | 39.0 | 1500 | 1.409 | 0.367 | 17.87 | 1.73 |
| No. 412... | Stainless chromium iron, annealed.................. | 1586 | 317 200 | 0.961 | 49.0 | 1500 | 1.755 | 0.450 | 20.85 | 2.02 |
| No. 413... | Free-Machining rustless steel. | 1245 | 249 000 | 0.754 | 25.0 | 800 | 0.900 | 0.239 | 12.60 | 1.22 |
| No. 416... | S.A.E. No. 1112, $1\frac{1}{4}$-in. square, cold-drawn, $\frac{1}{16}$-in. draft.... | 1475 | 295 000 | 0.745 | 24.5 | 1000 | 0.887 | 0.148 | 10.70 | 1.04 |
| No. 418... | S.A.E. No. 1120, $1\frac{1}{2}$-in. square, cold-drawn, $\frac{1}{16}$-in. draft.... | 1579 | 315 800 | 0.797 | 27.3 | 900 | 0.984 | 0.162 | 10.83 | 1.05 |
| No. 421... | S.A.E. No. 1315, 1-in. round, cold-drawn, $\frac{1}{16}$-in. draft... | 1561 | 312 200 | 0.789 | 26.5 | 860 | 0.955 | 0.144 | 10.35 | 1.00 |
| No. 422... | S.A.E. No. 6140, $1\frac{3}{16}$-in. round, cold-drawn, $\frac{1}{16}$-in. draft... | 1646 | 329 200 | 0.831 | 31.5 | 1250 | 1.138 | 0.220 | 11.36 | 1.10 |
| No. 426... | S.A.E. No. 1320, $1\frac{1}{2}$-in. round, cold-drawn, $\frac{1}{16}$-in. draft... | 1612 | 322 400 | 0.814 | 27.0 | 1050 | 0.975 | 0.161 | 11.12 | 1.08 |
| No. 50-A.. | Unleaded brass.............. | 1163 | 232 600 | 0 5875 | 30.0 | 1100 | 1.081 | 0.422 | 9.06 | 0.88 |
| No. 187... | Brass screw stock........... | 567 | 113 400 | 0.2863 | 8.75 | 325 | 0.316 | 0.047 | 5.14 | 0.50 |
| No. S3.... | Brass screw stock $1\frac{5}{16}$ in. diameter................ | 575 | 115 000 | 0.2902 | 8.0 | 300 | 0.288 | 0.057 | 4.65 | 0.45 |
| No. 33.... | Sheet brass, $\frac{3}{4}$ in. thick, half hard..................... | 595 | 119 000 | 0.3002 | 13.0 | 600 | 0.47 | 0.060 | 4.65 | 0.45 |
| No. 2RB.. | Red Brass.................... | 617 | 123 400 | 0.3117 | 10.75 | 500 | 0.389 | 0.054 | 4.55 | 0.44 |
| No. BB... | Bearing bronze............... | 503 | 100 600 | 0.2541 | 8.0 | 250 | 0.288 | 0.061 | 5.42 | 0.53 |
| No. MB... | Manganese bronze........... | 750 | 150 000 | 0.3789 | 23.0 | 1200 | 0.832 | 0.652 | 6.47 | 0.63 |
| No. GM... | Gun metal.................... | 564 | 112 800 | 0.2847 | 19.0 | 800 | 0.685 | 8.24 | 7.24 | 0.70 |
| No. Cu.... | Annealed copper............ | 1875 | 375 000 | 0.9467 | 50.0 | 1400 | 1.80 | 0.160 | 12.19 | 1.18 |
| No. N..... | Nickel "A"................... | 3140 | 628 000 | 1.586 | 65.0 | 2100 | 2.34 | 0.504 | 19.28 | 1.87 |
| No. M.... | Monel metal................. | 3335 | 667 000 | 1.684 | 41.0 | 1400 | 1.475 | 0.691 | 18.55 | 1.80 |
| No. Al-31 . | Aluminum alloy............. | 648 | 129 600 | 0.3270 | 11.0 | 600 | 0.398 | 0.058 | 3.85 | 0.37 |
| No. Al-12 . | Aluminum alloy............. | 552 | 110 400 | 0.2790 | 8.5 | 375 | 0.305 | 0.044 | 3.90 | 0.38 |
| No. DM... | Dow metal, type "E," cast... | 250 | 50 000 | 0.1264 | 5.0 | 250 | 0.180 | 0.041 | 2.66 | 0.26 |

[a] Values averaged for first 0.100 in. depth at 16 ft. per minute.

[b] Values averaged for body (below 0.100 in. depth) at 16 ft. per minute.

[c] First 0.100 in.

[d] Second 0.100 in.

[e] Depth of cut, first 0.125 in. from surface.

[f] Depth of cut, second 0.125 in. (between 0.125 and 0.250 in.).

[g] Composition Carbon 0.53, Manganese 0.52, Silicon 2.26, Sulfur 0.056, Phosphorus 0.32.

In summarizing these tests, the following information relative to tool size, shape, and the cutting conditions is given:

The planer tool was $\frac{1}{2}$-in. wide, end cutting with 15-deg. front-rake angle, depth of each cut was 0.010 in., speed 20 ft. per minute in all cases except 16 ft. per minute for malleable iron.

The $\frac{3}{4}$-in. drill had a speed of 152 r.p.m. and a feed of 0.012 in. per revolution and a helix angle of 30 deg.

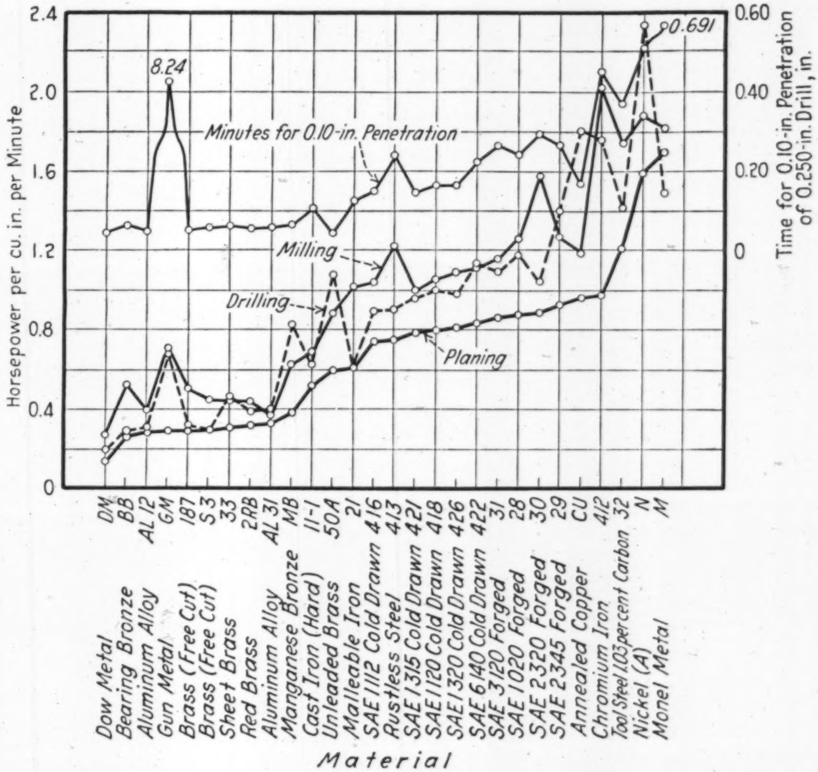

FIG. 11.—Showing a Comparison of Machineability Values for a Variety of Materials.

The $\frac{1}{4}$-in. drill had a helix angle of 24 deg. and was fed by a constant load of 93.75 lb.

Originally, penetration values were determined in inches per 100 revolutions, but later the readings were obtained more easily by taking the time in minutes to penetrate 0.100 in. For purpose of conversion $50\ pt = 1$, in which $p$ is the actual penetration per 100 revolutions and $t$ is the time in minutes to penetrate 0.100 in. when the drill rotates at 500 r.p.m. This conversion checks by actual experiment.

The milling cutter was 0.250 in. wide, end cutting, and had a 15-deg. front-rake angle; depth of cut 0.125 in., and feed 0.010 in. per cut.

In order to make the values shown in Table IV somewhat clearer, the values for horsepower per cubic inch of metal cut per minute by planing, drilling, and milling, as well as the time in minutes for the penetration drill to penetrate 0.100 in., are shown plotted in Fig. 11. The materials are arranged from left to right in order of steadily increasing power in planing. It was felt that this order gave the most uniform relation between the curves. While some exceptions to the order of materials are noted from the milling and drilling curves, the general tendency is seen to be rather consistent.

On this scale, it appears that malleable-iron bar No. 21 machines with slightly greater difficulty than cast iron bar No. 11-1, both in planing and milling. In drilling, it appears that the cast-iron bar machines with slightly greater difficulty. The differences, however, in planing and drilling are small. The cast-iron bar was cast in a section 1 in. square, much smaller than the malleable iron bar. The cast-iron bar, therefore, may be classed as comparatively hard, so that soft cast iron would appear much lower (to the left) in the scale of machineability.

# DISCUSSION ON MACHINEABILITY OF MALLEABLE IRON

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MR. H. A. SCHWARTZ.[1]—I should like to make a few comments on the subject of machineability, not particularly critical of Mr. Boston's paper, but to present a viewpoint or two. In his paper, Mr. Boston specifically said that he means by machineability tests a test of the tool-load type. Of course we should read an author's paper from the viewpoint of the scope in which he is working. I would suggest, however, as a matter of general consideration, that the thing one has to prove, to be of commercial use, is machineability in the sense of tool life. That is the only thing that the customer of the malleable foundry or the machiner of malleable castings values when he talks about machineability, and that raises the enormous difficulty of either making breakdown tests on the malleable iron, which seems to me impracticable, or demonstrating that tool load is a measure of tool life. I wish that could be done, but I think that so far such a relation rests more or less on assumption.

In connection with the hump type curve the author shows, we have observed the same thing with the end-mill type test; you can change the magnitude of the hump by the method of grinding the tool—grinding or honing or otherwise—so that the significance of such a hump has to be considered in the light of tool-edge conditions, and it is likely that caution must be used in the quantitative interpretation of such curves. I think our experience is not as accurate as that of Mr. Boston, but a lot of that work has been done. Results depend also on whether you cool or lubricate the tools, and how much.

As to the suggestion of the cause of falling off in load in the outside layers, we have seen a great decrease of load when the ferrite grains are oxidized on their boundaries. It is possible the outer layer is filled in with oxidized grain boundaries and that therefore one gets weak, then strong, then weak material.

Lastly, in that same connection, I would say a word of caution with respect to one type of curve shown in the paper. With regard to the distribution of carbon, in connection with the present meeting, we had some hope of presenting a paper on sampling for graphite. It proved impossible, however, to get that material ready in convincing shape in time for the meeting, but it may be well to point out that you cannot sub-divide a sample of malleable iron into small parts without losing carbon from it, and therefore the only accurate method I know of for obtaining correct data is to machine out a single piece at the location where you wish to analyze and dissolve that piece in double chloride solution and burn the residue, if you are going to draw quantitative conclusions from the values that are going to be used. The curves will be similar to those shown in the general form in which they will go up and down, and my thought is that you lose roughly about 0.25 per cent carbon.

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[1] Manager of Research, National Malleable and Steel Castings Co., Cleveland, Ohio.

MR. O. W. BOSTON[1] (*author's closure by letter*).—The author very much appreciates the remarks made by Mr. Schwartz. There are one or two points of his discussion which lead to comment.

There appears to be no information available in the literature on metal cutting which shows a consistent relation between tool life and the power required to cut a metal. It is recognized that the shop is interested primarily in tool life and not power. As Mr. Schwartz has pointed out, there are difficulties involved in making tool breakdown tests on malleable iron. The paper as presented gives the very first information accumulated by the author in an attempt to learn the characteristics of malleable iron castings as they affect machineability. Machineability is used here in a general way.

An attempt has been made to correlate the results of various specific methods of determining machineability with physical and chemical properties of the metal at various points below the surface. Additional results, along this line of study, not available at the time this paper was submitted, will be completed soon. It is hoped that from these studies it will be possible to undertake tool-life tests more logically and informatively.

It is shown here that to undertake a study of tool life, using depths of cut of any considerable magnitude, would lead to indefinite conclusions. The material is too variable for such tests, unless those tests are confined to definite strata of known characteristics.

The author does not concur in Mr. Schwartz's statement that the magnitude of the hump as shown can be changed by the method of grinding the planer tool, inasmuch as tests of this nature, conducted with the tool prepared in a number of different conditions of sharpness, showed practically no difference in results. The condition of the tool, however, was kept constantly sharp, in so far as physically possible. The two-lip counterboring tool used by the author is, in a way, similar to the end mill used by Mr. Schwartz. The author has repeatedly checked the results of the counterboring tool by running two tests in the same bar, and has found that the humps in all cases agree in themselves and with the humps obtained on the same bar with the planer tool. It is also significant that the hump as obtained by the milling cutter agrees closely with those obtained by the planer tool and the counterboring tool. The results of the milling cutter tests are appreciably susceptible to the degree of sharpness of the tool because of short intermittent cuts which continually change the size and shape of the built-up edge on the tool face. The author took chips comparatively heavy; 0.010 in. thick for the planer tool and 0.003 in. per tooth for the counterboring tool. Mr. Schwartz does not state the feed per revolution of the end mill used by him, nor does he give the number of teeth in the cutter. Presumably the feed per tooth was near to 0.001 in. On thin chips the influence of tool sharpness is more pronounced.

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[1] Professor, College of Engineering, University of Michigan, Ann Arbor, Mich.

Without a doubt, Mr. Schwartz is right when he states that the use of a lubricant or coolant will influence the results. It is known that a lubricant will influence the cutting force on a tool when it cuts one material more than when cutting another material. It follows that, when cutting a metal as variable as malleable cast iron, variations might be encountered.

Every precaution was made to prevent particles of the chip for each cut becoming lost prior to the chemical analyses. The complete chip of one stroke of the planer tool 0.010 in. deep, 0.5 in. wide, and 7 in. long was pushed into an envelope at the end of the cut. The complete contents of the envelope were then subjected to analysis.

It should be borne in mind that this paper contains only the initial information from machineability tests on malleable cast iron inaugurated by the Institute. The information given here is not final, but was all that was available for presentation at this meeting. It is hoped that progressively something of practical assistance to the users and makers of malleable iron will be developed.

# CORROSION OF MALLEABLE IRON

By F. L. Wolf[1] and L. A. Meisse[2]

## Introduction

Because malleable iron can be cast into intricate shapes with little foundry difficulty it has found a broad field for application. Its physical properties are usually well suited for application where relatively high tensile strength together with high shock resistance are required. Malleable iron castings are for this reason to be found in use in many locations and industries. There are instances where a high-grade casting is the only answer from a manufacturing standpoint. At other times, it is open to choice whether a forging, stamping or weld-assembled device is to be used. In both cases, the superiority of the materials will depend largely on their service record.

Among the most severe conditions to be met by these materials is corrosion in its various forms. It is customary to protect the iron from corrosion either with paints or protective metallic coatings, such as zinc or cadmium. Any superiority in regard to corrosion resistance of these materials will be reflected in the extent of their use, and for this reason we are endeavoring to establish relative corrosion values for the materials which fall in the same price range and which could, from a mechanical standpoint, be substituted for each other. Unfortunately, there are few data published regarding the corrosion of malleable iron. It is true that much malleable iron is used in applications where corrosion resistance is of very minor importance, as for instance, in the automobile industry and in tool machines, but for farm machinery, road building equipment, ship building and many others, there is need for definite information on the relative corrosion resistance of malleable iron compared to such material as forged steel or forged pure iron, with and without a certain amount of copper or other constituents. It is hoped that this paper will stimulate further researches by others interested in the field.

The tests described in this paper were conducted at the laboratories of The Ohio Brass Co. The company's diversified line of products allows many choices for materials, and the company is concerned not so much whether to build certain devices from malleable iron, forged steel or stampings, as that for the particular application a material is used which, from experience, is known to give a maximum service consistent with its cost.

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[1] Chief Engineer and Technical Superintendent, The Ohio Brass Co., Mansfield, Ohio.

[2] Research Metallurgist, The Ohio Brass Co., Mansfield, Ohio.

There are three types of corrosion which are encountered by these products. First, atmospheric corrosion in all conceivable locations; second, corrosion due to locomotive smoke; and, third, corrosion due to acid mine water. A certain amount of protective coating is applied to each of the articles. In some cases, paint has been found to suffice, whereas in others even a good galvanized coat has been found to be inadequate. The corrosion data reported in this paper deal, therefore, with these three types of corrosion. In the cases of smoke corrosion and mine-water corrosion, an accelerated test has been developed that is highly representative of the conditions which one would encounter in service, and the results can be obtained in much shorter time.

## LOCOMOTIVE SMOKE BOX CORROSION TESTS

By courtesy of the Pennsylvania Railroad we were permitted to use their smoke jacks on the locomotive round house at Mansfield, Ohio, for the tests. These smoke jacks were in continuous use, and therefore offered an ideal test ground for smoke corrosion. They were open on top so that rain could enter. The specimens were suspended in the smoke jacks and sets removed at intervals of two months for final testing. Along with the malleable iron and steels reported upon in this paper, a large number of other ferrous and non-ferrous materials were exposed. The results of these are to be reported later. While the test on this series of specimens is not fully completed, it has progressed far enough, having run for over two years, to give results which are ample for comparative evaluation. There are enough specimens exposed to extend the test up to a five-year period, and while it is not expected that contradictory evidence will be obtained at a later date, we are continuing this test for the sake of confirmation and accuracy.

### *Materials Tested:*

The following materials have been tested:

*Malleable Iron.*—The malleable iron series consisted of normal iron made in our own foundry as well as iron containing various copper contents, ranging from 0.25 to 2 per cent. A set of galvanized specimens of non-copper bearing malleable iron was also included. It has been found that the addition of copper to malleable iron does not materially influence its casting properties and physical properties. Contrary to general belief, it has been found that 2 per cent of copper can be alloyed with the iron without segregation of the copper. The copper-bearing alloys were made by placing the required amount of copper in the bottom of a hand ladle and tapping into the ladle. This gives enough stirring action to dissolve and thoroughly mix the copper. The specimens to be galvanized were first heated to a temperature of 1250° F. and quenched in water, which makes malleable iron immune to galvanizing embrittlement. Galvanizing was done at a temperature of 860° F., time of immersion three minutes. The excess zinc was shaken off by hand.

*Wrought Iron.*—The wrought iron analysis showed no copper and the material came in the hot-rolled condition.

*Basic Open-Hearth Steel.*—This material was also furnished in the hot-rolled condition.

*Commercial Pure Iron.*—The pure iron was a well-known commercial grade which we could obtain only in the form of round bars, and we therefore do not have comparative figures on the flat specimens.

TABLE I.—CHEMICAL AND PHYSICAL PROPERTIES OF THE MATERIALS AS RECEIVED.

| Designation | Chemical Composition, per cent | | | | | Tensile Strength, lb. per sq. in. | Yield Point, lb. per sq. in. | Elongation, per cent |
|---|---|---|---|---|---|---|---|---|
| | Carbon | Silicon | Phosphorus | Manganese | Copper | | | |
| Malleable Iron[a] | 2.61 | 0.82 | .... | 0.30 | .... | 45 500 | 33 450 | 8.7 |
| Malleable Iron[a] with 1 per cent Copper | 2.60 | 0.83 | 0.153 | 0.31 | 1.03 | 48 930 | 39 150 | 7.5 |
| 1021 Basic Open-Hearth Flat | 0.06 | 0.10 | 0.007 | 0.42 | nil | 58 150 | 44 150 | 34.2 |
| 3100 Basic Open-Hearth Round | 0.26 | 0.033 | 0.005 | 0.43 | nil | 66 500 | 44 200 | 36.0 |
| 1310 Wrought Iron Flat | 0.04 | 0.013 | 0.237 | 0.02 | nil | 51 500 | 37 900 | 20.6 |
| 1310 Wrought Iron Round | 0.05 | 0.015 | 0.093 | 0.02 | nil | 47 200 | 37 200 | 33.5 |
| 1021 Pure Iron Commercial Round | 0.04 | nil | 0.004 | 0.02 | nil | 44 850 | 31 300 | 40.0 |

[a] Analysis before annealing.

The chemical analysis of the materials reported in Table I appears to show a somewhat higher content of impurities than is generally accepted.

*Test Specimens:*

Both round and flat test specimens were used. For malleable iron, the round specimen was the A.S.T.M. standard tension test specimen for this material;[1] for all other materials, the round specimen was the A.S.T.M. standard specimen of 2-in. gage length.[2] All flat specimens were similar to the A.S.T.M. standard specimen of 8-in. gage length.[3]

The malleable iron specimens were exposed as cast and annealed, with the exception of those which have been galvanized. The rolled steel specimens were exposed in the received condition. For identification, the specimens were marked with a code number, which was

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[1] 1930 Book of A.S.T.M. Standards, Part I, p. 491.

[2] *Ibid.*, p. 70.

[3] *Ibid.*, p. 69.

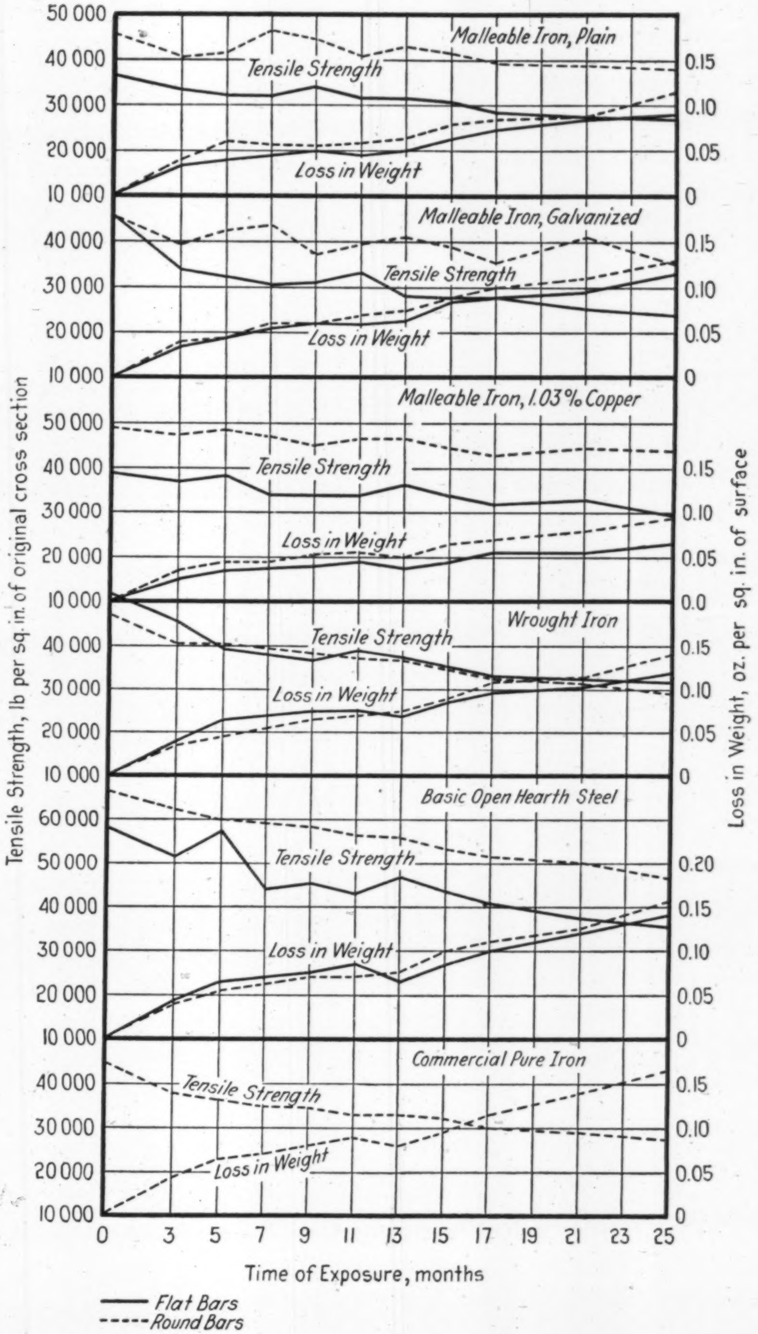


FIG. 1.—Showing Progress of Corrosion Losses and Its Effect on the Physical Properties of a Series of Test Specimens.

imprinted by means of drill marks. Flat specimens had an average surface area of 40 sq. in., and the round specimens $15\frac{1}{2}$ to 18 sq. in. A $\frac{3}{8}$-in. hole was drilled near the upper end of each bar, and the test specimens were strung on $\frac{1}{4}$-in. copper rod, separated from each other by porcelain ferrules. A sufficient number of sets of specimens was provided so that when one set was removed periodically from corrosive conditions, it could be cleaned, reweighed and broken with no necessity for returning it to the test. This procedure eliminates any masking of the corrosion rates which might occur due to removing corrosion products which otherwise would retard further attack on the base metal.

*Corrosion Losses Due to Locomotive Smoke:*

The curves in Fig. 1 show the progress of corrosion losses as they occurred in the series of test specimens. The curves also give the effect of corrosion loss on the physical properties of the bars. Some explanation of the method of reporting these figures might be necessary at this place.

The specimens after removal from the smoke box were cleaned by a light sand blasting and weighed. The difference between the original weight before exposure and the weight as obtained after corrosion divided by the number of square inches of original surface gives the loss figure expressed in unit surface, reported as ounces per square inch of original surface.

The physical properties were obtained by pulling the specimens in a 50,000-lb. Olsen testing machine. The loads required for tensile strength and yields point of the exposed bars are divided by the cross-section of the bar as determined before exposure. This gives a tensile strength, for instance, in pounds per square inch over the original cross-section exposed. The practical significance of such figures is that, as the material corrodes, its cross-section diminishes and while the tensile strength over the actual cross-section of sound material will be materially the same as in the specimen before exposure, its mechanical usefulness diminishes at the same rate as the cross-section becomes smaller.

The curves indicate that the rate of corrosion progresses uniformly. This fact is proved by the uniformity of the slope of the curves for loss in weight as well as the gradual decrease of the tensile strength when expressed over the original surface. In most instances a slight dip in the corrosion loss curve is noticed at the thirteenth month. This dip occurred when the first specimens were taken from the second smoke jack, due to differences in the corrosive conditions of the two smoke jacks used. The maximum of corrosion loss occurred on the samples of basic open-hearth steel and the pure iron follows next, then the wrought iron, and finally the malleable irons. Approximately 25 per cent increase in corrosion resistance has been found in malleable iron containing 1 per cent of copper. The malleable iron specimens, on removal from the smoke box, showed a tightly adhering scale, particularly on the copper-bearing specimens, and this has undoubtedly accounted in part for their higher corrosion resistance.

The following table gives the arithmetical mean of corrosion loss on the various irons, calculated from the loss data given in the curves.

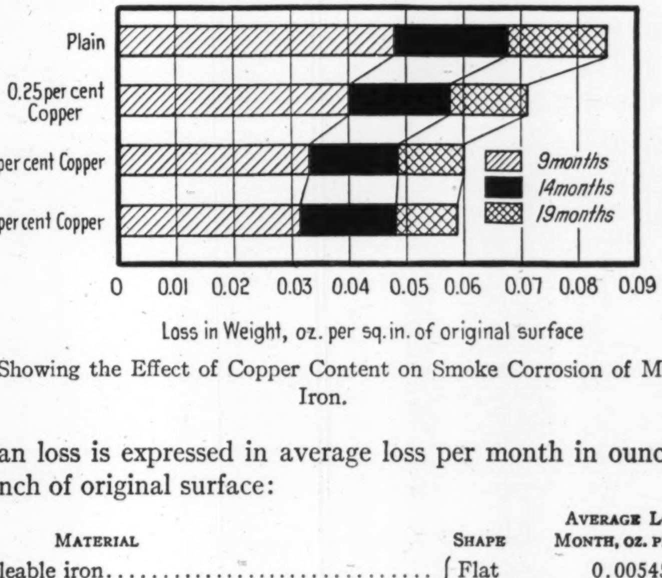


FIG. 2.—Showing the Effect of Copper Content on Smoke Corrosion of Malleable Iron.

The mean loss is expressed in average loss per month in ounces per square inch of original surface:

| MATERIAL | SHAPE | AVERAGE LOSS PER MONTH, OZ. PER SQ. IN. |
|---|---|---|
| Plain malleable iron | Flat | 0.00545 |
| | Round | 0.00650 |
| Malleable iron plus 1 per cent copper | Flat | 0.0044 |
| | Round | 0.0058 |
| Galvanized malleable iron | Flat | 0.00658 |
| | Round | 0.00718 |
| Wrought iron | Flat | 0.0076 |
| | Round | 0.00716 |
| Basic open hearth | Flat | 0.00855 |
| | Round | 0.00777 |
| Commercial pure iron | Round | 0.0087 |

## SMOKE BOX CORROSION OF COPPER-BEARING MALLEABLE IRON

Because of the superior corrosion resistance of copper-bearing malleable iron found in the test just described, it was decided to check the effect of variations in the copper content more thoroughly. Four heats of malleable iron which varied in chemical analysis were used in this experiment. Flat tension test bars were cast from the plain iron and an equal number were cast with additions of 0.25, 1 and 2 per cent of copper which was added to the ladle. Table II gives the physical properties of the round bars from these heats before corrosion testing.

The physical properties of the bars are low due to internal shrinkage. It will be noted that there is a slight tendency of the iron to gain in strength with increase in copper content.

TABLE II.—PHYSICAL PROPERTIES OF MALLEABLE IRON SPECIMENS BEFORE CORROSION TESTING.

| Heat | Tensile Strength, lb. per sq. in. | Yield Point, lb. per sq. in. | Elongation, per cent |
|---|---|---|---|
| | NO COPPER | | |
| No. 749 | 40 500 | 32 000 | 7.0 |
| No. 750 | 38 300 | 30 600 | 4.7 |
| No. 751 | 45 300 | 31 300 | 9.4 |
| No. 752 | 48 200 | 36 300 | 12.5 |
| Average | 43 075 | 32 550 | 8.4 |
| | 0.25 PER CENT COPPER | | |
| No. 749 | 42 900 | 35 900 | 6.3 |
| No. 750 | 39 100 | 34 500 | 5.5 |
| No. 751 | 43 800 | 31 300 | 7.8 |
| No. 752 | 47 500 | 32 200 | 10.9 |
| Average | 43 325 | 33 475 | 7.6 |
| | 2 PER CENT COPPER | | |
| No. 749 | [a] | [a] | [a] |
| No. 750 | [a] | [a] | [a] |
| No. 751 | 46 400 | 37 600 | 7.8 |
| No. 752 | 48 200 | 38 000 | 7.8 |
| Average | 47 300 | 37 800 | 7.8 |

[a] No bars tested.

The corrosion tests were conducted in the smoke box of the Pennsylvania Railroad, the arrangement being identical to that of the previous test with the exception that the bars after removal for inspection, cleaning and weighing were returned to the exposure for further tests. Figure 2 shows the average loss in weight of the various heats plotted according to copper content.

It will be seen that as the copper content increases, the corrosion loss materially decreases. It seems that with 2 per cent of copper a second constituent starts to form, so it is doubtful whether a further increase in copper is warranted. With 2 per cent of copper, an average decrease in corrosion resistance of 25 per cent is noted.

## ACID MINE-WATER TESTS

In connection with an investigation on the protective value of paints and similar protective coatings on malleable iron in acid mine water, we included malleable iron containing various percentages of copper. It would have been desirable to have comparable tests available on other materials along with malleable iron, but since this investigation was conducted to evaluate protective coatings rather than the base material, the results secured are only incidental to this test.

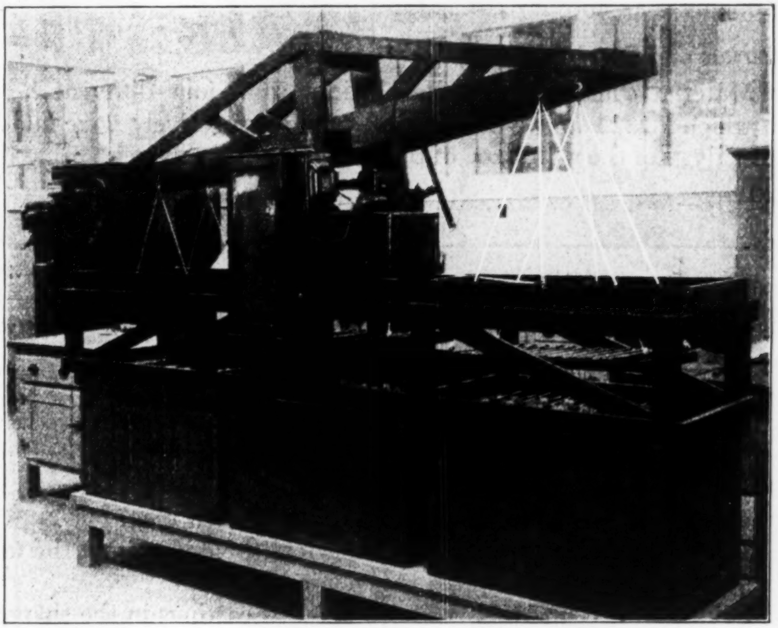


FIG. 3.—Apparatus Used in Mine-Water Tests.

The acid mine water, which was made up synthetically, had the following composition (similar to that found in the Pittsburgh district); according to Bulletin No. 4, of the Carnegie Institute of Technology, page 9. Sample No. 8 clear.

| | PARTS PER MILLION |
|---|---|
| $H_2SO_4$ | 701 |
| $H_2SO_3$ | 1567 |
| $Fe_2O_3$ | 401 |
| $Al_2O_3$ | 431 |
| $CaO$ | 127 |

*Testing Apparatus:*

Earthenware crocks each having a capacity of 45 gallons served as containers into which the specimens were dipped. The capacity of the container was sufficient to avoid any considerable change in the acidity of the solution during the test as evidenced by repeated analysis of the corrosion medium. Figure 3 illustrates the general arrangement of the apparatus. The test consisted of alternately submerging the specimens in the water and withdrawing and drying them. A complete cycle per hour consisting of one-half hour in the water, and one-half hour out of the water, gave sufficient time to thoroughly soak and dry the specimens.

*Materials and Specimens:*

Material consisted of malleable iron from four different heats into which 0.25, 1 and 2 per cent of copper respectively were added. The analysis of the white iron of the various heats is given in Table III.

TABLE III.—ANALYSIS OF MATERIAL AND TEST RESULTS OF CORROSION IN MINE WATER.

| Heat | Analysis, per cent | | | Loss in Weight after 500 Immersions, per cent | | | |
|---|---|---|---|---|---|---|---|
| | Carbon | Manganese | Silicon | No Copper | 0.25 per cent Copper | 1 per cent Copper | 2 per cent Copper |
| No. 749......... | 2.64 | 0.29 | 0.78 | 5.75 | 5.35 | 6.26 | 5.5 |
| No. 750......... | 2.90 | 0.28 | 0.82 | 5.71 | 5.6 | 5.8 | 5.37 |
| No. 751......... | 2.78 | 0.31 | 0.80 | 4.97 | 5.75 | 5.63 | 6.4 |
| No. 752......... | 2.70 | 0.26 | 0.70 | 4.74 | 5.3 | .... | 5.8 |

Specimens consisted of standard A.S.T.M. tension test bars which were supported by nichrome wires from the wooden rack.

The ratio of the corrosion resistance malleable iron from the four heats, with and without copper, is given in Table III.

Using test specimens in the form of small castings in the shape of mine hangers, the following average losses for irons of various copper contents after 500 immersions were obtained:

| | LOSS IN WEIGHT, PER CENT |
|---|---|
| No copper.............................. | 5.40 |
| 0.25 per cent copper...................... | 5.75 |
| 1 per cent copper ...................... | 6.30 |
| 2 per cent copper...................... | 7.15 |

These results indicate that there is no advantage in using copper-bearing malleable iron for acid mine-water service, since with increase in copper content the corrosion rate increases. Incidentally, it was found that galvanized malleable iron was immediately attacked by the mine water with a resultant rapid disintegration of the zinc coat.

After completion of the test, the galvanized specimen, nevertheless, showed less corrosion loss than ungalvanized ones.

## EFFECT OF BASE MATERIAL ON ZINC COATING

Since galvanizing has been found most effective as a protective coating on ferrous materials, a relative evaluation of the protective action of such zinc coatings as influenced by the base material is of importance. We have found repeatedly that when malleable iron and rolled steels, both galvanized at the same temperature and in the same zinc bath, are exposed together under identical corrosive condition, the malleable iron is greatly superior to the steels. Figure 4 shows a suspension clamp used in connection with an electrified section of the Boston & Maine Railroad. One of the specimens was removed after a service of seventeen years, whereas the other one has not been in service. The malleable iron clamps have almost the same dimensions, indicating very little corrosion loss in the exposed clamp.

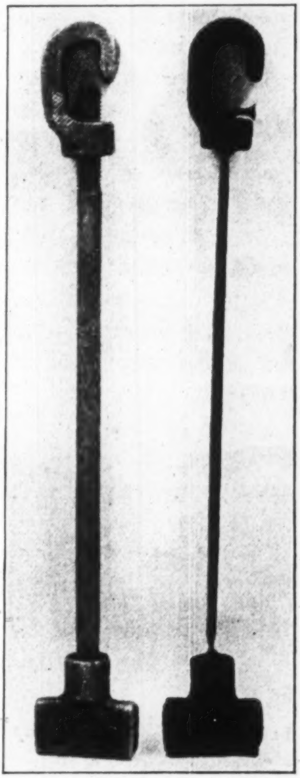
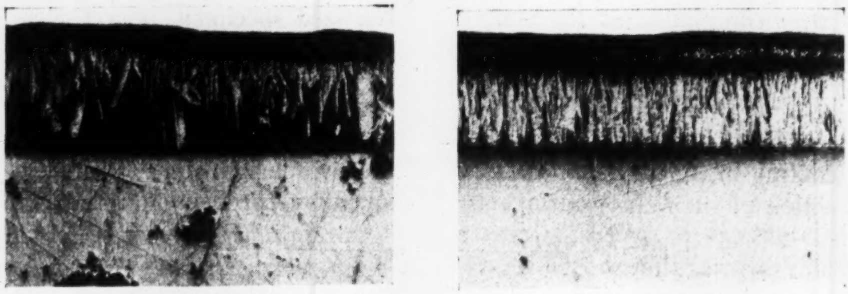
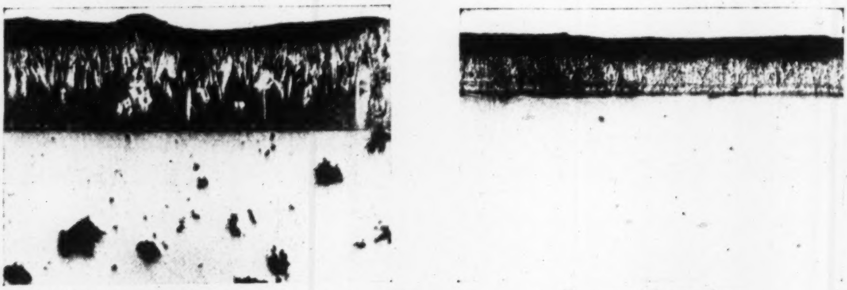


FIG. 4.—Showing a New Suspension Clamp and a Similar Type Suspension Clamp After Being in Service for Seventeen Years.



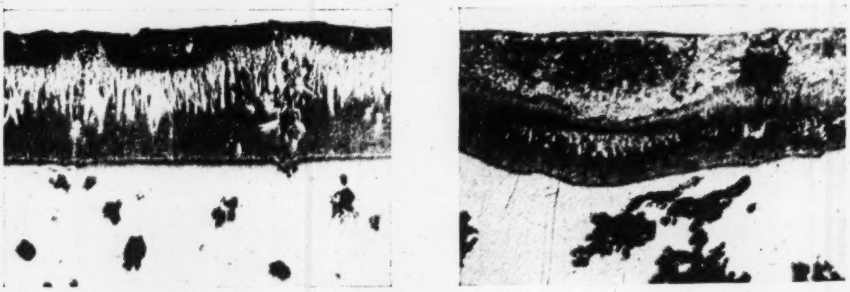
Plain malleable iron.

Basic open hearth.



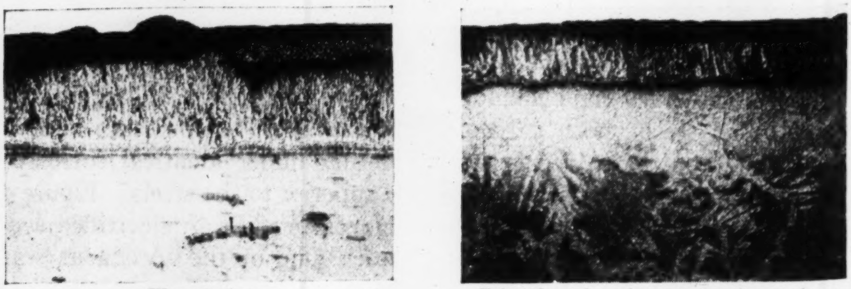
Malleable iron with 1 per cent copper.

Commercial pure iron.



Malleable iron with 2 per cent copper.

Taken from malleable iron insulator cap.



Wrought iron.

Taken from forged copper bearing steel cap.

FIG. 5.—Photomicrographs of Sections Through Zinc Coatings (× 100). Etched with 2-per-cent nitric acid in alcohol for 10 seconds.

The steel supporting rod, however, may be seen to have corroded to a dangerous extent. While such outstanding differences in corrosion between the malleable iron and steel have not been found by the semi-service tests reported above, it appears that the thickness or quality of the galvanized coating must influence the corrosion rate for such big differences to result. In order to test this assumption we have taken specimens of identical compositions to the ones exposed in the smoke box, consisting of malleable iron with and without various percentages of copper, wrought iron, basic open-hearth steel and pure iron. The surfaces were ground and all of the specimens galvanized at the same time. The galvanizing temperature was 860° F. and the time of immersion three minutes.

Microsections of the zinc coat were then prepared after applying a heavy deposit of electrolytic copper over the zinc. The photomicrographs in Fig. 5 show representative sections of the coating. Particularly noticeable is the thick iron alloy layer on the malleable iron specimens which contributes materially to the corrosion resistance. While we cannot make the general statement that the thickness of the alloy layer is the chief factor which affords protection to galvanized ferrous materials, the test conducted at the Bureau of Standards and reported in the *Journal of Research,* Vol. I, supports this assumption. If we deal with highly corrosive conditions, as for instance locomotive smoke or mine water, this conclusion is undebatable.

## CONCLUSIONS

The following conclusions are presented:

1. Under the action of locomotive smoke, malleable iron shows a decided superiority over wrought iron, basic open-hearth steel and commercial pure iron.

2. Copper additions in malleable iron up to 2 per cent have been found to materially increase its resistance to corrosion in locomotive smoke.

3. Copper additions to malleable iron decrease its resistance to corrosion in acid mine water.

4. Malleable iron without copper protected by a hot-dip galvanized coating shows a decided superiority over steel similarly protected after a service of seventeen years in an atmosphere contaminated by smoke.

5. A hot-dip galvanized malleable iron, with and without copper, shows a heavier iron-zinc alloy layer than either wrought iron, basic open-hearth steel or commercial pure iron, when galvanized under identical conditions.

# DISCUSSION ON CORROSION OF MALLEABLE IRON

Mr. J. S. Vanick.[1]—I should like to call attention to the table of corrosion data upon the various materials on page 427 of the paper by Messrs. Wolf and Meisse, showing that galvanized malleable iron under the conditions stated is corroding some 10 per cent faster than the plain uncoated malleable iron. I wonder whether we are to infer from this that galvanizing increases the corrosion rate in this particular iron, or whether there has been an initial protection culminating in an acceleration in corrosion rate over the period of the test, in the case of the galvanized product?

Mr. L. A. Meisse.[2]—In most of the tables given in the paper we have mentioned only plain malleable iron without any protective coating, since when a protective coating is applied the rate of corrosion is materially altered. Thus, malleable iron of a particular composition with a definite rate of corrosion in a given medium will have a higher initial corrosion rate if another element like zinc is applied to the surface. This difference in rate of corrosion continues until the pure zinc is eaten away. This is the reason why the galvanized specimens reported in our tests have shown a higher rate of corrosion, although we know that malleable iron protected by galvanizing will last longer in service than without such protection.

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[1] Research Metallurgist, International Nickel Co., Bayonne, N. J.

[2] Research Metallurgist, The Ohio Brass Co., Mansfield, Ohio.

# ACCELERATED FREEZING AND THAWING AS A QUALITY TEST FOR CONCRETE AGGREGATES

By F. C. Lang[1] and C. A. Hughes[2]

## Synopsis

The purpose of the investigation described in this paper was to develop a definite test for the quality of gravel pebbles for use in exposed concrete that would eliminate the personal equation which is so large a factor in the methods now available. For this purpose an accelerated freezing-and-thawing test consisting of 5 minutes in hot water, 5 minutes in ice water, and 30 minutes directly submerged in calcium chloride brine at 0° F. was used. While the majority of tests were made on this cycle, data are given on other accelerated and slow cycles of freezing and thawing.

The results of accelerated tests (10 cycles per day) are given for 10, 20, and 30 cycles of freezing and thawing on gravel pebbles from eighteen different sources, two sedimentary rocks, and nine deleterious materials. Similar data are given for concretes made with seventeen of the above materials. Methods of evaluating the effect of freezing and thawing are described and compared. Data are given on some of the many variables which must be taken into consideration in the development of a satisfactory freezing-and-thawing test and on which little information is yet available. Further research is required before such a test can be used as a basis for specification.

The purpose of this investigation was to obtain information for specifying the quality of gravel pebbles as coarse aggregate for concrete subjected to severe exposure as in pavements and bridge piers in northern climates. The general practice in the past has been to define quality by the abrasion test and by limitation of the quantity of deleterious materials. The authors are of the opinion that the abrasion test within wide limits is not a measure of the durability of aggregates and that the selection of deleterious materials depends too much on local individual opinions resulting in considerable confusion among engineers and producers of aggregates. A specific test with suitable test limits would provide a definite unit for measuring the permissible amounts of deleterious materials and it would then not be necessary for various engineers to hand-pick samples and select what in their opinion were the various objectionable substances.

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[1] Professor of Highway Engineering, University of Minnesota, and Engineer of Tests and Inspection, Minnesota Department of Highways.

[2] Associate Professor of Structural Engineering, University of Minnesota.

In order for such a test to be practical it should be completed within a day or two. For that reason and because in northern climates freezing and thawing is the most destructive agency, an accelerated freezing-and-thawing test was considered as the most promising way of determining the quality of the aggregates. Various freezing-and-thawing cycles and methods of measuring the results have been tried. The authors realize that sufficient data have not as yet been obtained for specification purposes and expect to continue the investigation.

## MATERIALS

The coarse aggregates included gravels from eighteen different plants, two crushed stones, and nine selected materials which would be classified as deleterious. All the gravels, with one exception (BB), have been used within the past few years on highway construction in Minnesota. The nature of the deleterious materials is believed to be generally known. The material referred to as "chocolate bars" consisted of secretions having a soft interior, usually clay, and a hard ferruginous or manganitic shell. Limestone No. 1 contained clay seams. Limestone No. 2 was high in clay, and is locally referred to as soapstone.

The fine aggregate used in all concrete specimens was from a large washing plant. It has been used in concrete for many years and is known to be of good quality. The fineness modulus was 2.85.

The cement was a well-known brand of standard portland cement of medium strength and fineness.

## FREEZING-AND-THAWING CYCLES

Various cycles of freezing and thawing were employed as follows:

*Cycle G.*—This cycle consisted of 5 minutes in water at 200 to 212° F., 5 minutes in water at 35 to 40° F., and 30 minutes in calcium chloride brine at 0° F. ± 10° F.; 40 minutes per cycle, 10 cycles per day.

*Cycle H.*—This cycle was similar to cycle G with the exception that the period of immersion in the calcium chloride brine was 15 minutes instead of 30 minutes and with the addition of 1 hour under water at a pressure of 150 lb. per sq. in.; 85 minutes per cycle, 5 cycles per day.

*Cycle O.*—This cycle consisted of 3½ hours in air at 0° F. ±10°, and alternately 1 hour and 16 hours submerged in water at room temperature (70° F.); 2 cycles per day.

*Cycle S.*—This cycle consisted of 8 hours at room temperature and 16 hours at 0° F. ±10° F. frozen submerged in water, 1 cycle per day.

*Special Cycles.*—Similar to cycle G but with varying periods in freezing solution.

### SIZE OF SPECIMENS

*Coarse Aggregate.*—For cycle G, each test specimen consisted of 1500 g. each of $1\frac{1}{2}$ to 1-in., 1 to $\frac{1}{2}$-in., and $\frac{1}{2}$ to $\frac{1}{4}$-in. material (round-hole screens), a total of 4500 g. per specimen. This grading corresponds to the average grading of the specimens as received.

For cycle S, each specimen consisted of 3200 g. of $1\frac{1}{2}$ to $\frac{3}{4}$-in. and 1300 g. of $\frac{3}{4}$ to $\frac{3}{8}$-in. material (square-mesh screens), a total of 4500 g. per specimen.

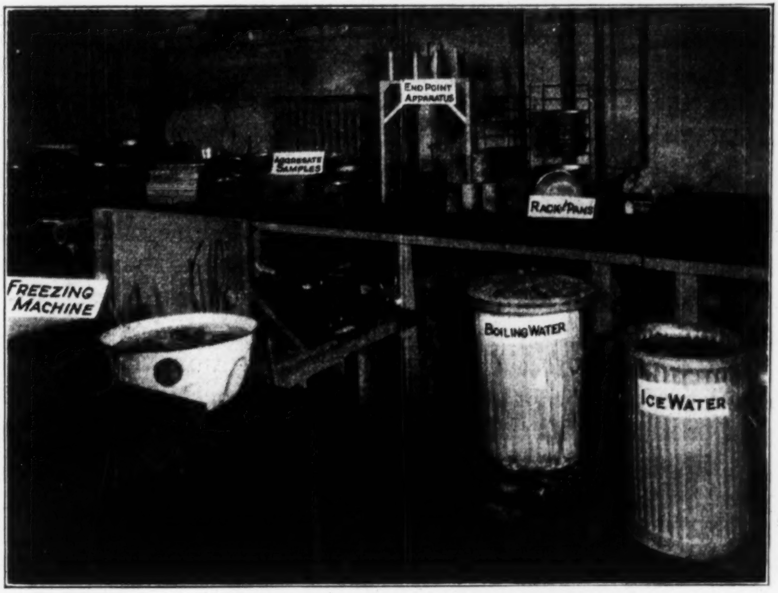

FIG. 1.—Apparatus Used in Freezing-and-Thawing Tests.

In the special cycles the specimen consisted of 1500 g. each of $1\frac{1}{2}$ to 1-in. and $\frac{1}{2}$ to $\frac{1}{4}$-in. material, a total of 3000 g. per specimen.

*Concrete.*—Cylinders $2\frac{3}{4}$ in. in diameter and $5\frac{1}{2}$ in. high were used in all tests. This small size of specimen was chosen in order to reduce the load on the freezing machine and so expedite the work.

### TEST PROCEDURE

*Coarse Aggregate:*

For cycle G and the special cycles the fractions of each specimen after washing and drying were sieved for 10 minutes on a Rotap before testing, and after every 10 cycles. This introduced a certain amount of abrasive action which is considered to be desirable. All samples were tested on the same stack of nine standard square-mesh sieves, varying from the 1-in. to the No. 100 sieve.

At the end of every 10 cycles the specimens were dried, sieved and then washed on the No. 12 sieve. The freezing-and-thawing test was continued only on the portion retained on the No. 12 sieve. The calculations for percentage loss, however, were based on the original dry sample. The amounts of fine material lost after each tenth cycle were taken into consideration in computing sieve analyses. Differences between dry weights before and after each 10 cycles were considered to be due to material finer than the No. 100 sieve washed out of the sample during the cycle.

For the cycle S tests, the specimens were dried after each 10 cycles and the amounts passing the $\frac{3}{8}$-in. sieve determined.

The apparatus used in these tests is shown in Fig. 1. The freezing machines are 8-hole Nizer ice-cream freezers, two units being used. Four holes in each machine were filled with the brine solution. The aggregate specimens were held in pans with 50-mesh screens on the bottoms and were covered with perforated metal plates, each size in a separate pan. The pans were held in racks, 6 pans or 2 specimens to the rack. Due to the heavy load resulting from the accelerated cycle, it was only possible to use one rack per machine. These were placed in successive holes each cycle, each hole being used every 4 cycles, in order to maintain as uniform a temperature as possible. The ice water was used primarily to reduce the freezing load. In addition it probably increases the absorption and therefore the effectiveness of the test.

### *Concrete:*

The proportions of the concrete by absolute volumes were 1:2.33:2.92 with a water-cement ratio of 5.0 gallons per sack, except for a few specimens made with a water-cement ratio of 4.90 gallons per sack. While the grading was constant for all aggregates, the differences in shape and surface characteristics resulted in some variation in consistency, but all mixes were considered workable. The maximum size of coarse aggregate was limited to $\frac{3}{4}$ in. All specimens were cured in the moist room until tested.

The end point was determined by a simple flexure test made on the apparatus shown in Fig. 1. The central load was such as to develop an apparent extreme fiber stress of 10 lb. per sq. in. calculated on the original nominal diameter of the specimens. The specimens were subjected to this test only when their appearance was such as to indicate that they were near the end point. Usually only two or three such trials were required.

## DATA AND DISCUSSION

Preliminary tests had indicated that the type of cycle, particularly in regard to the rate of freezing and the duration of the freezing period had considerable effect on the ratings that might be given various aggregates, especially if of widely different characteristics such as limestone and sandstone. Some data from these preliminary tests have been given in a discussion by the authors on a paper by F. H. Jackson, "Relation Between Durability of Concrete and Durability of Aggregates."[1] However, in the hope that a specification might be developed in time for 1931 construction, what appeared at that time to be the most suitable cycle, cycle G, was chosen for the main series of tests.

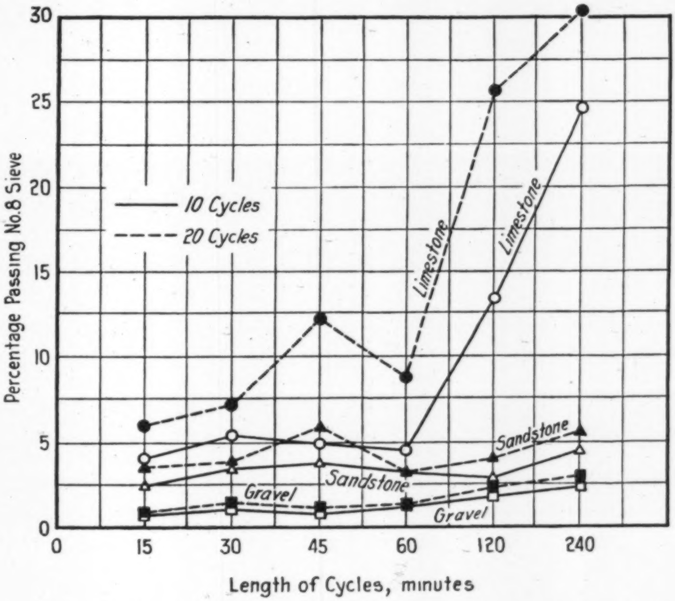

FIG. 2.—Effect of Duration of Freezing Period on Different Types of Coarse Aggregate.

**Note break in scale at 60 and 120 minutes.**

Data from tests made subsequently to this decision are shown in Fig. 2. Here the only variable from cycle G is the duration of the freezing period. It will be noted that the effect of the different periods of immersion was quite small on the sandstone and gravel compared to that on the limestone for the longer periods of freezing. Similar data were obtained in the preliminary tests and in other

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[1] ***Proceedings*, Highway Research Board, Vol. 10, December, 1930.**

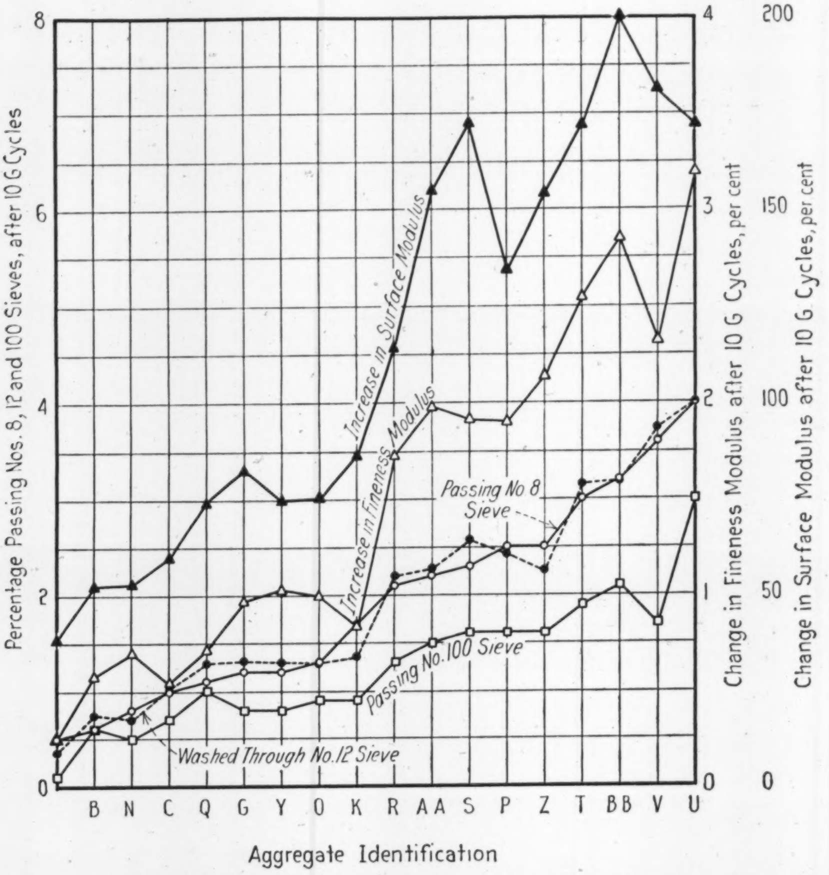

FIG. 3.—Relation Between Different Methods of Measuring the Effect of Freezing and Thawing on Coarse Aggregates.

tests now in progress with specimens frozen submerged either in brine or water. This figure is also a good illustration of the difficulties experienced in attempting to formulate standard test procedure for freezing and thawing. Irregularity in the data such as is shown for limestone for the cycle having the 60-minute period is attributed to variation in the quality of the specimens rather than indicative of any peculiarity of the particular cycle.

Having chosen the cycle, it was next necessary to decide on the method of measuring the effect of freezing and thawing. In Fig. 3 are shown the data obtained by the methods which were selected as the most practical. Since the five methods give data in general agreement and since the percentage passing the No. 8 sieve could be obtained simply and easily, this was chosen as a basis for rating the aggregates. The percentage change in surface modulus is regarded as the best theoretical measure because of its wide spread and the fact that it emphasizes the products of disintegration. However, it involves rather lengthy calculations and is largely affected by the percentage passing the No. 100 sieve which by our test procedure was obtained by difference and was therefore of questionable accuracy.

TABLE I.—DATA FROM FREEZING-AND-THAWING TESTS OF AGGREGATES, PERCENTAGE PASSING NO. 8 SIEVE, CYCLE G.

| Aggregate Identification | Specimen | 10 Cycles | | | | 20 Cycles | | | | 30 Cycles | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1½ to 1 in. | 1 to ½ in. | ½ to ¼ in. | 1½ to ¼ in. | 1½ to 1 in. | 1 to ½ in. | ½ to ¼ in. | 1½ to ¼ in. | 1½ to 1 in. | 1 to ½ in. | ½ to ¼ in. | 1½ to ¼ in. |
| U | 1 | 1.2 | 3.2 | 4.6 | 3.0 | 1.9 | 5.1 | 6.9 | 4.7 | 9.8 | 7.5 | 7.7 | 8.4 |
| | 2 | 7.5 | 5.4 | 4.2 | 5.6 | 10.0 | 7.4 | 6.6 | 8.1 | 10.9 | 8.4 | 7.3 | 8.3 |
| | 3 | 1.8 | 3.3 | 5.5 | 3.5 | 2.7 | 4.7 | 7.2 | 4.5 | 4.5 | 6.5 | 8.4 | 6.3 |
| | Average | 3.5 | 4.0 | 4.8 | 4.0 | 4.9 | 5.7 | 6.9 | 5.8 | 8.4 | 7.5 | 7.8 | 7.7 |
| T | 1 | 2.2 | 2.4 | 3.8 | 2.9 | 3.2 | 4.0 | 5.1 | 4.1 | 3.7 | 4.3 | 5.7 | 4.6 |
| | 2 | 3.8 | 3.6 | 3.6 | 3.6 | 5.7 | 5.0 | 4.8 | 5.3 | 7.6 | 5.8 | 6.0 | 6.4 |
| | 3 | 0.5 | 1.2 | 5.8 | 2.5 | 0.9 | 2.2 | 6.4 | 3.2 | 1.3 | 3.5 | 8.3 | 4.4 |
| | Average | 2.2 | 2.4 | 4.4 | 3.0 | 3.3 | 3.7 | 5.4 | 4.2 | 4.2 | 4.5 | 6.7 | 5.1 |
| BB | 1 | 2.1 | 2.5 | 3.4 | 2.9 | 3.8 | 5.0 | 6.0 | 5.0 | 4.6 | 5.0 | 9.3 | 6.4 |
| | 2 | 3.1 | 3.2 | 4.3 | 3.6 | 6.5 | 6.3 | 6.7 | 6.5 | 7.7 | 8.1 | 8.6 | 8.1 |
| | 3 | 2.0 | 3.4 | 4.2 | 3.2 | 5.8 | 5.6 | 5.1 | 5.5 | 7.4 | 7.4 | 8.0 | 7.6 |
| | Average | 2.4 | 3.0 | 4.0 | 3.2 | 5.4 | 5.6 | 5.9 | 5.7 | 6.6 | 6.8 | 8.6 | 7.4 |
| V | 1 | 2.2 | 3.8 | 6.2 | 4.0 | 4.1 | 4.9 | 7.9 | .6 | 4.7 | 4.9 | 9.1 | 6.2 |
| | 2 | 0.8 | 2.6 | 5.0 | 2.8 | 1.3 | 4.5 | 7.7 | 4.5 | 3.2 | 5.4 | 8.1 | 5.7 |
| | 3 | 2.8 | 3.0 | 6.4 | 3.9 | 3.7 | 4.7 | 7.9 | 5.3 | 6.1 | 6.0 | 10.9 | 7 7 |
| | Average | 1.9 | 3.1 | 5.9 | 3.6 | 3.0 | 4.7 | 7.8 | 5.1 | 4.7 | 5.4 | 9.4 | 6.5 |
| R | 1 | 2.5 | 2.1 | 2.1 | 2.2 | 3.5 | 3.1 | 3.5 | 3.3 | 3.8 | 3.9 | 4.2 | 3.9 |
| | 2 | 1.4 | 1.4 | 2.9 | 2.0 | 3.6 | 2.6 | 4.6 | 3.5 | 4.3 | 3.0 | 5.3 | 4.1 |
| | Average | 1.9 | 1.7 | 2.5 | 2.1 | 3.5 | 2.8 | 4.0 | 3.4 | 4.0 | 3.4 | 4.7 | 4.0 |

The effect of cycle G as measured by the percentage passing the No. 8 sieve is shown in Fig. 4. Here the aggregates are arranged in order of the percentage loss for the total sample of 4500 g. In reading this figure one should be careful to take the breaks in scale into account. The points of significance are considered to be as follows:

1. The large increase in loss for the deleterious materials and the rather wide differences shown by the aggregates. Aggregate H has

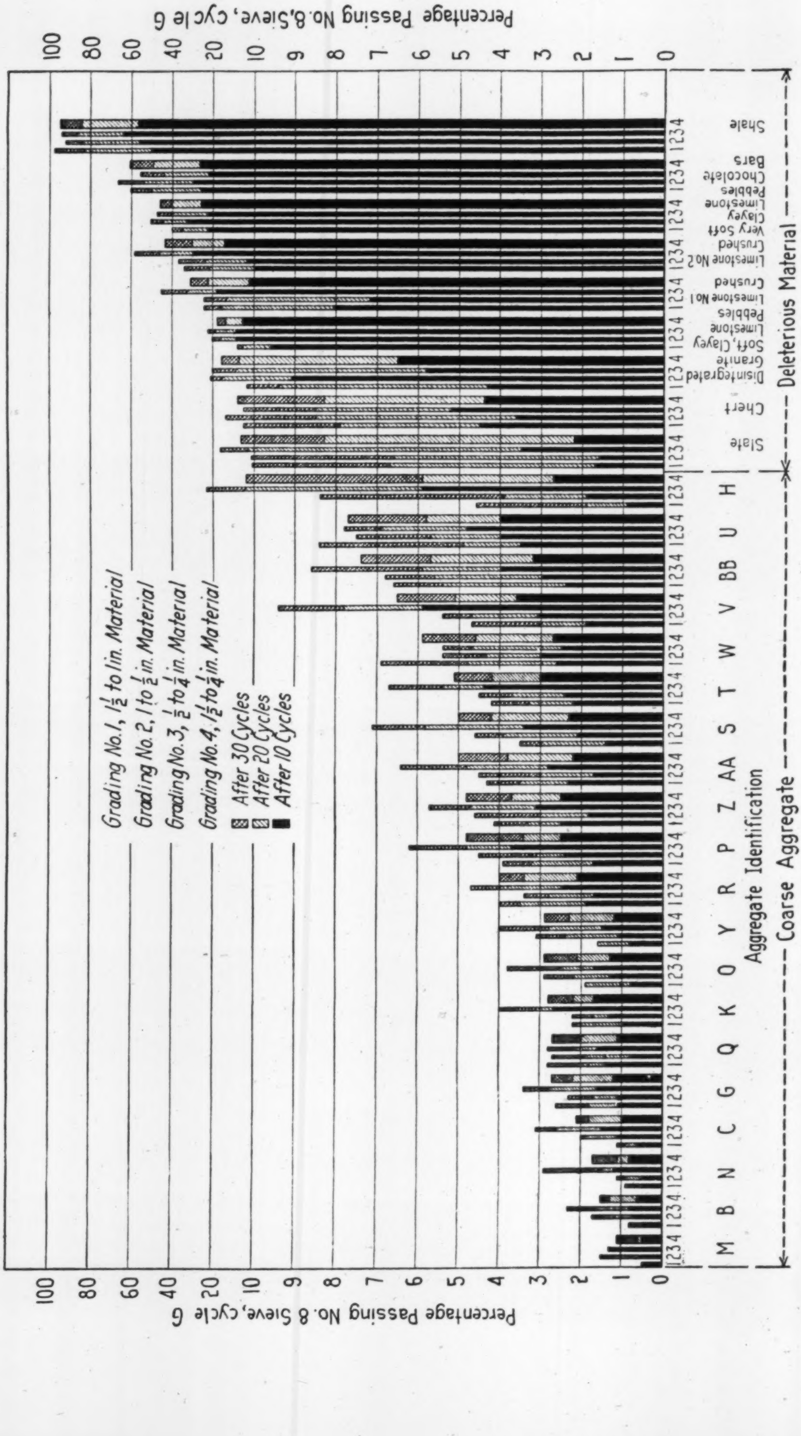

FIG. 4.—Percentage Passing No. 8 Sieve at 10, 20, and 30 Cycles (Cycle G), Arranged in Order of the Percentage Passing at 30 Cycles.

**Note break in scale at 10 and 20 per cent.**

been regarded generally as a doubtful material and from these tests it would appear that it should be classified as a deleterious material (see also Figs. 2 and 9).

2. While there is fair agreement between the ratings given by the 20- and 30-cycle data, the rating on the 10-cycle data would be considerably different, notably for the fine-grained sedimentary materials such as material H and slate, these being Nos. 17 and 12 on the tenth cycle, and 20 and 21 on both the twentieth and thirtieth cycles. Apparently such materials will show marked disintegration either on increasing the number of cycles or on increasing the duration of the freezing period.

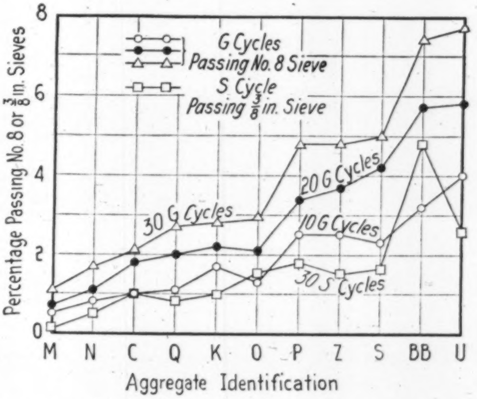

FIG. 5.—Relation Between 10, 20, and 30 Accelerated Cycles (Cycle G) and 30 Slow Cycles (Cycle S) of Freezing and Thawing.

3. Similar differences in rating would result from omitting one or two fractions from each specimen. This has been done in the preparation of Fig. 9 for which only the 1 to ¼-in. material was considered.

4. In general, the smaller-sized fractions showed greater losses than the larger-sized fractions. This may be due either to the greater effect of the cycle on the smaller sizes or to a greater percentage of soft material in these sizes.

All the above observations point to the importance of the size of specimen and the number of cycles on the classification of the aggregate.

Data from tests of five aggregates on which repetitions were made are given in Table I. These show the agreement that may be expected from tests made on material from the same source and are also typical of the data plotted in Fig. 4.

The agreement shown in Fig. 5 between the data obtained from the 30 cycles of cycle S and from cycle G, particularly after 20 and 30 cycles, is regarded as the best evidence submitted as to the suitability of the accelerated cycle as a measure of the resistance of

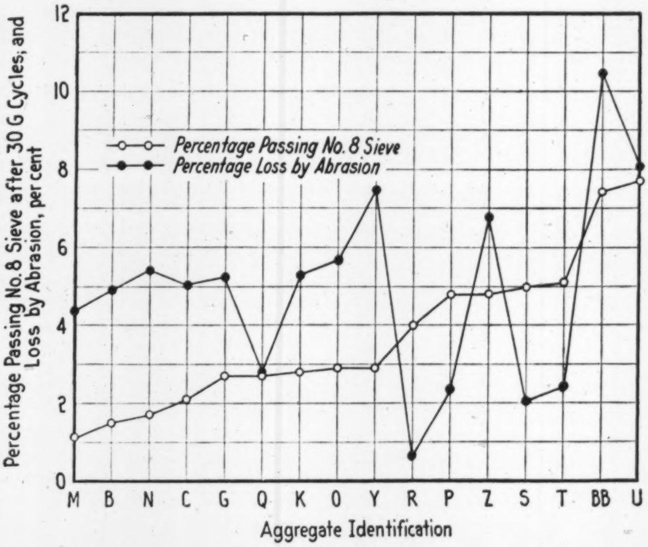

FIG. 6.—Relation Between Abrasion Loss and Percentage Passing No. 8 Sieve After 30 Cycles (Cycle G) of Accelerated Freezing and Thawing.

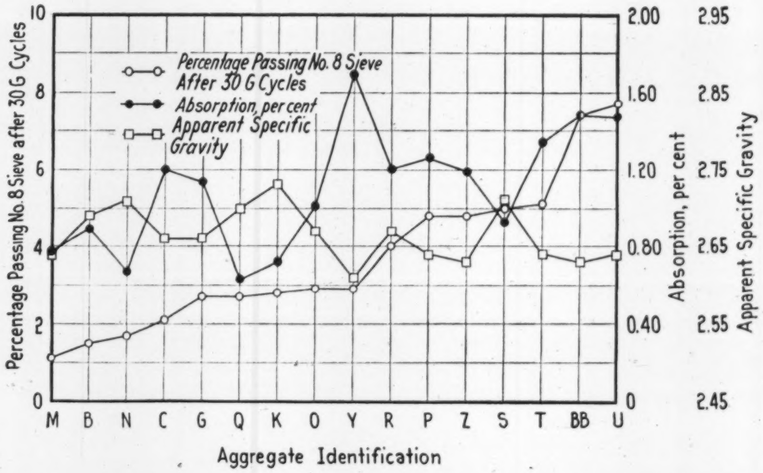

FIG. 7.—Relation Between Percentage Absorption, Apparent Specific Gravity and Percentage Passing No. 8 Sieve After 30 Cycles (Cycle G) of Accelerated Freezing and Thawing.

aggregates to freezing and thawing. It will be recalled that in cycle S the specimens were frozen submerged in water, 1 cycle per 24 hours, whereas in cycle G they are frozen submerged in calcium chloride brine, 1 cycle every 40 minutes. The cycle S tests were completed about six months before the other tests reported herein were contemplated and were made on different samples. Aggregate BB, showing the greatest variation between cycle S and cycle G, was sampled from test pits for the cycle S tests and from the plant product for the cycle G tests.

Figure 6 shows that there is little, if any, agreement between abrasion loss and the disintegration resulting from freezing and thawing. This is to be expected since the abrasion test measures properties other than those required in a durable aggregate and also depends to a large extent on the shape of the particles. The abrasion data were obtained from routine control tests, each point representing three or more tests made within the last year.

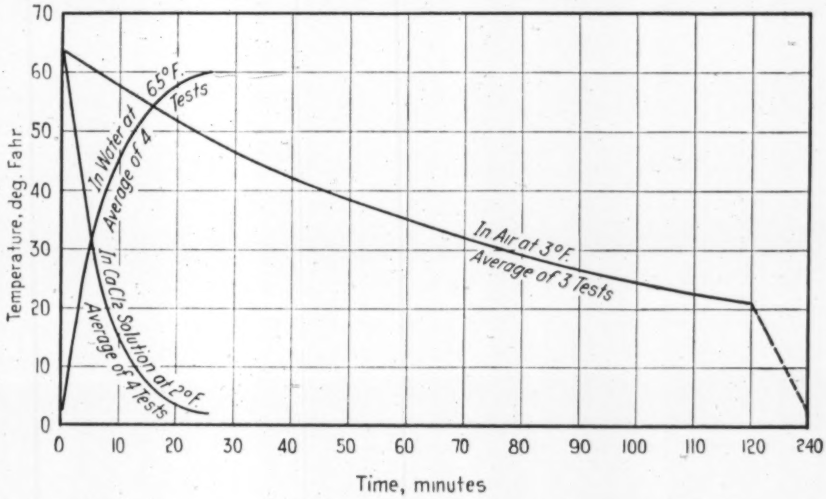

FIG. 8.—Time-temperature Relations for $2\frac{3}{4}$ by $5\frac{1}{2}$-in. Concrete Cylinders. Note break in scale at 120 minutes.

Data on absorption and apparent specific gravity shown in Fig. 7 were also obtained from averages of control tests. As was anticipated, there appears to be no relation between the apparent specific gravity of different materials and their resistance to freezing and thawing. On the contrary much better agreement was looked for with absorption than that shown. The trend of the absorption is in agreement with the freezing-and-thawing test in that the high values of absorption accompany the high percentages passing the No. 8 sieve. The presence of a small amount of deleterious materials in a sample will be much more apparent in the percentage loss through a No. 8 sieve than in the absorption test since one or two pieces may very appreciably raise the percentage passing a No. 8 sieve while the effect on the absorption of the total sample would be comparatively small. For example, the clayey limestone No. 2 had an absorption of 6 per cent whereas the loss at 30 cycles through the No. 8 sieve was 44 per cent. That is to say, the effect of a given percentage of this material in the freezing-and-thawing test is about 7 times its effect on absorption. Consideration of this fact indicates that the absorption test cannot be expected to be a very reliable criterion of the quality of heterogeneous materials such as gravels.

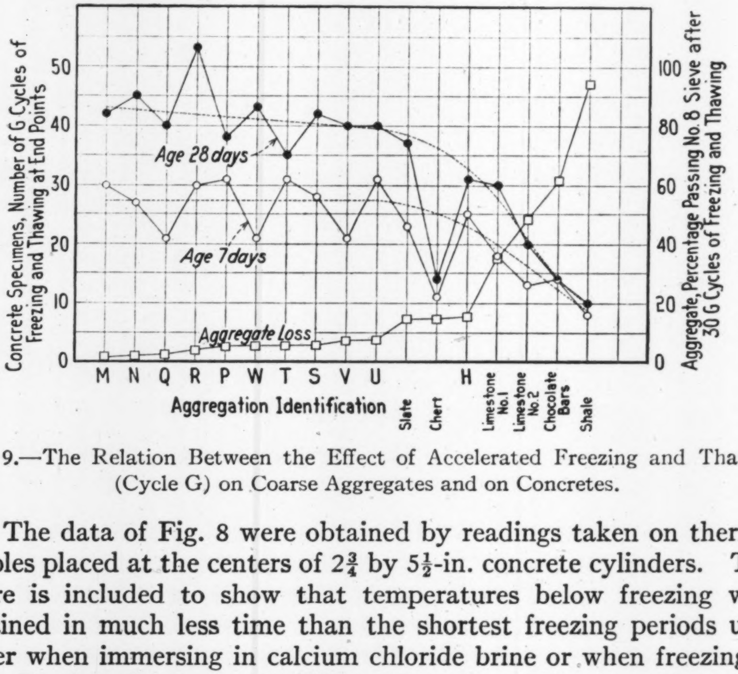

FIG. 9.—The Relation Between the Effect of Accelerated Freezing and Thawing (Cycle G) on Coarse Aggregates and on Concretes.

The data of Fig. 8 were obtained by readings taken on thermocouples placed at the centers of $2\frac{3}{4}$ by $5\frac{1}{2}$-in. concrete cylinders. This figure is included to show that temperatures below freezing were obtained in much less time than the shortest freezing periods used either when immersing in calcium chloride brine or when freezing in air. It also shows the rapidity of the change in temperature on direct immersion.

Freezing-and-thawing data on aggregates are of little value if correlation cannot be shown with the resistance of concretes made with the same aggregates. Data from tests of concrete specimens are given in Table I. Since the maximum size of aggregate used in the cylinders was $\frac{3}{4}$ in., the aggregates are rated on the average of the percentages passing the No. 8 sieve after 30 cycles for the 1 to

TABLE II.—DATA FROM FREEZING-AND-THAWING TESTS OF CONCRETE CYLINDERS.

**Mix—1:2.33:2.92 by absolute volumes.**
**Water-cement ratio—*a* specimens, 4.90 gal. per sack; *b, c, d* specimens, 5.0 gal. per sack.**

| Aggregate Identification | 1 to ½-In. Aggregate, Percentage Passing No. 8 Sieve | | | Specimen | Cycle G | | Cycle H | | Cycle O—28 days | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Number of Cycles at End Point | | Number of Cycles at End Point | | Number of Cycles at End Point | Condition of Specimens at 115 Cycles |
| | 10 Cycles | 20 Cycles | 30 Cycles | | 7 days | 28 days | 7 days | 28 days | | |
| M | 0.7 | 0.9 | 1.1 | a | 37 | 67 | .. | .. | .. | O.K. |
| | | | | b | 37 | 49 | .. | .. | .. | O.K. |
| | | | | c | 27 | 45 | .. | .. | .. | Several pops |
| | | | | d | 27 | 33 | .. | .. | .. | Pop on end |
| | | | | Average b, c, d specimens | 30 | 42 | | | | |
| N | 0.8 | 1.2 | 2.0 | b | 28 | 56 | 23 | 67 | .. | O.K. |
| | | | | c | 37 | 47 | .. | .. | .. | O.K. |
| | | | | d | 17 | 31 | .. | .. | .. | Pop on side |
| | | | | Average b, c, d specimens | 27 | 45 | | | | |
| Q | 1.2 | 2.0 | 2.7 | b | 21 | 47 | 22 | 72 | .. | O.K. |
| | | | | c | 17 | 41 | .. | .. | .. | O.K. |
| | | | | d | 25 | 32 | .. | .. | .. | Pop on corner |
| | | | | Average b, c, d specimens | 21 | 40 | | | | |
| R | 2.1 | 3.4 | 4.0 | a | 30 | 37 | .. | .. | .. | Pop on end |
| | | | | b | 30 | 53 | .. | .. | .. | O.K. |
| P | 3.0 | 3.6 | 5.3 | b | 28 | 40 | 20 | 40 | .. | O.K. |
| | | | | c | 36 | 44 | .. | .. | .. | Four pops on side |
| | | | | d | 30 | 28 | .. | .. | .. | Pop on side |
| | | | | Average b, c, d specimens | 31 | 38 | | | | |
| W[a] (Sandstone) | 2.7 | 4.5 | 5.4 | a | 52[b] | 76[b] | .. | .. | .. | O.K. |
| | | | | b | 45[b] | 76[b] | 25 | 84 | .. | O.K. |
| | | | | c | 16 | 38 | .. | .. | .. | O.K. |
| | | | | d | 25 | 49 | .. | .. | .. | O.K. |
| | | | | Average c, d specimens | 21 | 43 | | | | |
| T | 3.4 | 4.5 | 5.6 | b | 39 | 34 | 16 | 60 | .. | Four small pops on side |
| | | | | c | 29 | 41 | .. | .. | .. | O.K. |
| | | | | d | 24 | 29 | .. | .. | .. | Four small pops |
| | | | | Average b, c, d specimens | 31 | 35 | | | | |
| S | 2.7 | 4.8 | 5.8 | a | 32 | 61 | .. | .. | .. | |
| | | | | b | 35 | 48 | 21 | 76 | .. | Pop on side |
| | | | | c | 29 | 49 | .. | .. | .. | O.K. |
| | | | | d | 20 | 32 | .. | .. | .. | Pop on side |
| | | | | Average b, c, d specimens | 28 | 42 | | | | |
| V | 4.5 | 6.2 | 7.4 | b | 19 | 52 | .. | .. | .. | O.K. |
| | | | | c | 26 | 39 | .. | .. | .. | Pop on side |
| | | | | d | 18 | 28 | .. | .. | .. | Seven pops on sides |
| | | | | Average b, c, d specimens | 21 | 40 | | | | |

[a] Crushed stone; all other aggregates identified by a letter are gravel pebbles.
[b] Not averaged.

TABLE II.—*Continued.*

Mix—1:2.33:2.92 by absolute volumes.
Water-cement ratio—*a* specimens, 4.90 gal. per sack; *b* *c*, *d* specimens, 5.0 gal. per sack.

| Aggregate Identification | 1 to ¼-in. Aggregate, Percentage Passing No. 8 Sieve | | | Specimen | Cycle G | | Cycle H | | Cycle O—28 days | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Number of Cycles at End Point | | Number of Cycles at End Point | | Number of Cycles at End Point | Condition of Specimens at 115 Cycles |
| | 10 Cycles | 20 Cycles | 30 Cycles | | 7 days | 28 days | 7 days | 28 days | | |
| U.......... | 4.4 | 6.3 | 7.6 | a | 35 | 49 | .. | .. | .. | Not rated, poor specimen. |
| | | | | b | 31 | 40 | .. | .. | .. | Large spall on corner, 3 or 4 pops on side. |
| Slate....... | 2.5 | 9.1 | 14.5 | b | 27 | 46 | 14 | 66 | .. | O.K. |
| | | | | c | 26 | 37 | .. | .. | .. | Aggregate exposed on corners and ends. |
| | | | | d | 16 | 27 | .. | .. | .. | |
| | | | | Average b, c, d specimens | 23 | 37 | | | | |
| Chert....... | 4.4 | 8.5 | 14.8 | b | 9 | 17 | .. | .. | 105 | End point, cracking at 7 cycles. |
| | | | | c | 15 | 12 | .. | .. | .. | 3 or 4 pops on side, near end point. |
| | | | | d | 10 | 13 | .. | .. | .. | Big expansion, numerous bulgess. |
| | | | | Average b, c, d specimens | 11 | 14 | | | | |
| H[a] (Limestone, Top Ledge) | 3.6 | 8.0 | 15.6 | b | 29 | 29 | 18 | 50 | .. | Aggregate failing at ends, 5 pops on side. |
| | | | | c | 26 | 39 | .. | .. | .. | Popping on corners, checking on sides. |
| | | | | d | 19 | 26 | .. | .. | .. | Four pops on side. |
| | | | | Average b, c, d specimens | 25 | 31 | | | | |
| Limestone No. 1 (Soapstone) | 13.8 | 25.2 | 35.2 | a | 23 | 27 | .. | .. | .. | |
| | | | | b | 15 | 36 | .. | .. | 114 | End point, cracking at 69 cycles. |
| | | | | c | 26 | 32 | .. | .. | .. | Five pops on sides, surface badly checked. Considerable expansion near end point. Cracking at 80 cycles. |
| | | | | d | 13 | 21 | .. | .. | 105 | End point. |
| | | | | Average b, c, d specimens | 18 | 30 | | | | |
| Limestone No. 2 Bottom Ledge) | 21.1 | 35.5 | 48.3 | b | 11 | 26 | 11 | 30 | .. | Badly disintegrated, close to end point. |
| | | | | c | 16 | 19 | .. | .. | 110 | End point, cracking at 75 cycles. |
| | | | | d | 12 | 14 | .. | .. | 106 | End point. |
| | | | | Average b, c, d specimens | 13 | 20 | | | | |
| Chocolate Bars | 26.4 | 49.6 | 61.6 | b | 14 | 14 | .. | .. | 46 | End point, popping at fourth cycle. Cracking at tenth cycle. |
| Shale....... | 60.5 | 85.7 | 93.8 | b | 7 | 9 | 8 | 8 | 90 | End point. |
| | | | | c | 7 | 14 | .. | .. | 96 | End point. |
| | | | | d | 9 | 6 | .. | .. | 75 | End point. |
| | | | | Average b, c, d specimens | 8 | 10 | | | | |

¼-in. material only. This results in a somewhat different order of aggregates than that in Fig. 2 in which the order was based on the percentage loss for the 1½ to ¼-in. material. While it was recognized that the effect of the quality of the aggregate would be more pronounced at later ages when the cement paste had attained greater resistance, the desire to obtain results at the earliest possible moment led to the cylinders being tested after 7 and 28 days of moist curing.

Figure 9 shows the relation between the accelerated freezing-and-thawing test on the coarse aggregates and on the corresponding concrete cylinders having a water-cement ratio of 5.0 gallons per sack of cement. As shown in Table II each end point, with the exception of those for aggregate R, W, U and chocolate bars, is the average of three separate tests. The specimens for each test were made on different days and tested in separate racks. Considering the non-uniformity of the particles in a sample and the many details involved in a freezing-and-thawing test, the agreement between individual specimens is regarded as satisfactory.

The dotted lines of Fig. 9 represent what the authors consider to be the trend of the end points. For the 7-day concrete specimens no difference is apparent until the aggregate loss is more than 10 per cent while on the 28-day specimens the number of cycles to the end point appears to decrease as the aggregate loss increases. The concrete specimens for both the 7- and 28-day curing periods show a very marked decrease in resistance to freezing and thawing when aggregates which may be classed as deleterious were used. It will be noted that the method used for measuring the quality of the aggregate did not indicate the extent to which chert is detrimental in concrete. The chert used contained pieces which expanded and thus disrupted the concrete but similar pieces in the aggregate test did not break in such a manner that a very large percentage would pass a No. 8 sieve.

Figure 9 also shows the beneficial effect of improved quality of the cement paste resulting from longer curing. It will be noted, however, that the end points at 7 and 28 days for the cylinders made from deleterious materials disintegrating with pronounced expansion such as chert, chocolate bars, and shale are very close together indicating that little assistance can be expected from the paste when such aggregates are used. It will be seen also from Table II that somewhat greater resistance is shown by the *a*-cylinders which have a slightly lower water-cement ratio.

Other tests made at this laboratory and elsewhere have shown the accelerating effect from more complete saturation prior to freezing than is given by ordinary submersion in water. With the expectation that the combination of fast freezing and greater saturation would give a faster cycle than either used separately, a few cylinders were run on the cycle H. From Table II it will be seen that the end points for the cycle H tests at 7 days are in general considerably lower whereas the end points at 28 days are higher than those for the cycle G tests. When these specimens were started the marked influence of the period of immersion in the brine solution (see Figs. 2 and 10) was not known. Apparently for the 28-day specimens the accelerating effect of the pressure treatment was more than offset by the decrease in the period of immersion (15 minutes in cycle H, 30 minutes in cycle G).

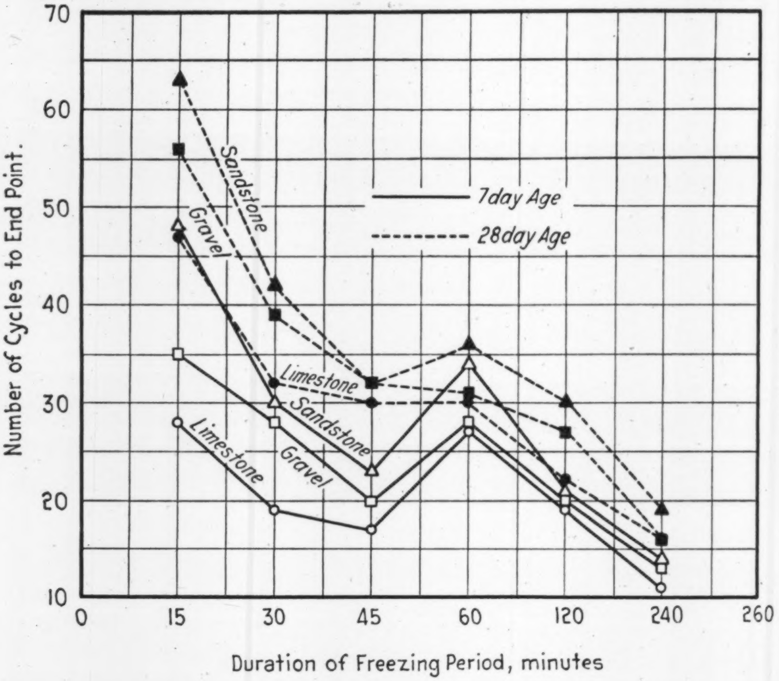

FIG. 10.—Effect of the Duration of the Freezing Period on $2\frac{3}{4}$ by $5\frac{1}{2}$-in. Concrete Specimens Made with Different Types of Coarse Aggregate.

Note break in scale at 60 and 120 minutes.

Since the method of freezing and thawing used in the accelerated cycles appears to be so widely different from the freezing and thawing occurring naturally in the field, it was thought desirable to test some cylinders by a method that would approximate field conditions. For this reason, specimens for cycle O tests were made, these all being 28 days old at the start of the test. The result of a visual inspection of these specimens at 115 cycles, or their end points, is given in Table II. While it is impossible to show any definite relation between cycle H and cycle G, the data are in general agreement.

Figure 10 shows the effect of various periods of immersion on concrete specimens made from the aggregates for which similar data are given in Fig. 2. Each point shown is the average of three tests. As was the case with the aggregates the destructive effect of freezing and thawing increases with the duration of the freezing period. These data again show the greater durability resulting from longer curing. To facilitate the work and for convenience in testing, all specimens for each age and for each period of immersion were made and tested at the same time. The peak in the graphs shown for the 60-minute immersion period for the 7-day specimens is probably due to some undetected irregularity in test procedure. From the wider spacing of the graphs for the different aggregates at the shorter periods of immersion, it would appear that these periods are more suitable for detecting differences in aggregate quality or paste quality. The end points of all specimens whether tested at 7 or 28 days tend to converge at the longer periods of freezing. This is believed to indicate that disintegration for these longer freezing periods depends more on the quality of the paste than the quality of the aggregate, particularly when the large difference between the loss shown by the limestone and the losses by the other two aggregates at the 120- and 240-minute cycles are taken into consideration.

## CONCLUSIONS

1. While further research is required, there appears to be a distinct possibility of developing an accelerated freezing-and-thawing test which will provide a more reliable and definite measure of the quality of gravel aggregates than methods of testing now in use.

2. Of the numerous cycles used by the authors, cycle G is believed to be the most desirable for testing aggregates.

3. The grading of the aggregate specimen and the type of cycle, particularly in regard to rapidity of cooling and duration of freezing period, have an important effect on the disintegration resulting from freezing and thawing.

4. To be assured of properly rating fine-grained sedimentary rocks such as limestone and slate, it is necessary to use more than 10 cycles of cycle G or longer freezing periods than 30 minutes. The rating of the other materials tested was much less affected by the number of cycles and the duration of the freezing period.

5. Data obtained on gravel pebbles by a slow cycle (24 hours) of freezing and thawing in water correlated closely with that obtained by an accelerated cycle (40 minutes) consisting of 5 minutes in hot

water, 5 minutes in ice water, and 30 minutes freezing directly immersed in brine solution.

6. A relationship exists between the durability of coarse aggregates and the durability of concretes made therefrom.

7. The cement paste of concrete specimens used in freezing and thawing tests intended to reveal the durability of the aggregate should be of such quality at time of test that disintegration will depend more on aggregate quality than paste quality.

8. From the data presented it would appear that 10 per cent passing the No. 8 sieve after 30 cycles of cycle G is the limit for aggregates suitable for use in concrete for exposed situations.

# DISCUSSION

MR. C. H. SCHOLER[1] (*by letter*).—The authors are to be complimented upon the very excellent progress they have made toward developing a satisfactory method for quickly determining the resistance of aggregates to frost action. Ignorance of this property of the aggregates proposed for use has resulted in many badly damaged structures and in the rejection of good material which was considered of questionable quality but is really of a quality entirely satisfactory from the point of view of weathering characteristics.

The authors' conclusion that the abrasion test is of no value in measuring the weathering ability of an aggregate is unquestionably sound. The writer's experience indicates that none of the common tests of aggregates are of any assistance in this respect.

The writer has for several years been making freezing-and-thawing tests of concrete and aggregates.[2,3] The apparatus used in these tests is in most respects similar to that used by the authors. The chief variation has been in the method of exposure to the freezing-and-thawing conditions. The very sudden temperature changes introduced by the author in cycle G would seem to be such as to produce a shattering effect rather than failure due to crystal growth in the pores of the aggregate. The writer would question whether the boiling water was essential in this cycle. Thawing will occur very rapidly in water at normal temperature and, on some work the writer has done, seems to have about the same effect upon unsound materials as does the hot water. Unless absolutely necessary to secure the rapid results, the writer would suggest eliminating the use of hot water as it introduces another variable—that of sudden temperature change.

Referring to the concrete tests, the writer would like to point out that his experience shows that freezing-and-thawing tests of concrete at ages under 60 days should not be made where normal cement is used. The resistance of the mortar to this type of disintegration is changing very rapidly during the first 60 days and except for very unsatisfactory aggregate, the mortar would probably fail first. The writer would also question the advisability of using boiling water in the thawing cycle for the concrete. It is known that boiling very markedly lowers the resistance of concrete to freezing and thawing[1] and other physical properties are also altered.

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[1] Professor of Applied Mechanics, Kansas State College of Agriculture, Road Materials Laboratory, Manhattan, Kans.

[2] C. H. Scholer, "Some Accelerated Freezing and Thawing Tests on Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 472 (1928).

[3] C. H. Scholer, "Durability of Concrete," *Proceedings*, Highway Research Board, Vol. 10, December, 1930.

MR. H. F. GONNERMAN[2] (*by letter*).—The paper by Lang and Hughes presents valuable information on an accelerated freezing-and-thawing test which appears promising as a means of testing aggregates for soundness. Anyone who has had experience in tests of this nature will recognize that there are many difficulties to be overcome before a suitable method can be developed. The most favorable features of the method described appear to be its simplicity and the rapidity with which results can be obtained. The rather well-defined correlation brought out in Fig. 9 of the paper between the failure of the aggregate when tested alone and when tested in concrete is to the writer the most significant feature of the results presented.

Another significant feature is what appears to be a general agreement in the results for gravel aggregates when tested by the accelerated cycle (cycle G) and by the slow cycle (cycle O). This points to the possibility of the development of an accelerated test which will give results comparable to those to be expected under the conditions of exposure encountered in the field.

There are a few points in the accelerated test of the authors that might be given further study. Direct immersion in calcium chloride brine does not seem to be as good as immersion out of contact with the brine. On the aggregates alone there probably would be but very little, if any, chemical action when the time of contact is considered, but on the other hand, concrete cylinders would appear to be acted upon to their detriment. It might, therefore, be better to substitute alcohol for the brine, or else enclose the samples in containers which are themselves immersed in the brine.

The practice of immersing the specimens for five minutes in boiling water might be given further consideration. Subjecting concrete specimens repeatedly to such treatment would tend to bring about cracking and failure because of the sudden temperature changes. Further work on this point would serve to clear up any misgivings. While it may not prove to be advisable to eliminate the boiling water treatment entirely, the severity of its effects could be materially lessened by thawing the specimens in water at room temperature, and then chilling them for a few minutes in ice water before freezing.

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[1] W. J. Schlick, "Effects on Concrete of Immersion in Boiling Water and Oven Drying," *Bulletin 59*, Iowa State College Engineering Experiment Station.

[2] Manager, Research Laboratory, Portland Cement Assn., Chicago, Ill.

The recent trend toward more rigid inspection and specifications for aggregates makes it highly desirable that methods of testing aggregates for soundness be further developed and, above all, standardized so that comparable results may be obtained by different laboratories. A standard method of test would permit a better evaluation to be made of the quality of the aggregates entering into concrete structures, particularly concrete roads and other structures which are to be exposed to severe climatic conditions. The results obtained by the authors indicate that progress toward this end is being made.

MESSRS. F. C. LANG[1] AND C. A. HUGHES[2] (*authors' closure by letter*).—Discussion of several points raised by Messrs. Scholer and Gonnerman has been given previously by the authors in discussion of a paper by F. H. Jackson, "Relation Between Durability of Concrete and Durability of Aggregates.[3] The authors there expressed themselves as being doubtful of the advisability of direct immersion in brine solution for mortars and concretes and have since abandoned this procedure. However, direct immersion in brine is still being used for aggregates. Except for a few tests made with carbon tetrachloride, the authors have experimented with no freezing liquids other than calcium chloride brine. The accelerated cycle now used for mortars and concretes consists of a thawing period submerged in water at room temperature and at a pressure of 150 lb. per sq. in. and a freezing period in air at 0° F. ± 10° F. This cycle results in a reasonably rapid disintegration of a nature very similar to that observed in the field.

The results of a few tests to determine the effect of alternations of hot (207° F. ± 5° F.) and cold water (42° F. ± 5° F.) on aggregates are given in the following table:

| | Percentage Loss Through No. 8 Sieve | | | | | |
|---|---|---|---|---|---|---|
| | Alternations in Hot and Cold Water | | | Cycle-G | | |
| | Sieving | 10 Cycles | 20 Cycles | Sieving | 10 Cycles | 20 Cycles |
| Sandstone W | 1.47 | 2.29 | 2.86 | 1.25 | 2.7 | 4.6 |
| Gravel P | 0.41 | 0.60 | 1.23 | 0.40 | 2.5 | 3.4 |
| Limestone No. 2 | 0.56 | 1.05 | 1.21 | 0.65 | 17.5 | 31.1 |

[1] Professor of Highway Engineering, University of Minnesota, and Engineer of Tests and Inspection, Minnesota Department of Highways.

[2] Assistant Professor of Structural Engineering, University of Minnesota.

[3] *Proceedings*, Highway Research Board, Vol. 10, December, 1930.

The effect of ten minutes of sieving on a rotap and corresponding data for cycle-G are given in the same table. The effect of sieving can only be determined from the sieve analyses made prior to freezing and thawing. Assuming that there is some further loss due to subsequent sievings, the effect of temperature alternations of the character used have but little effect on the aggregates tested. It is possible that submersion of frozen material in hot water such as occurs in cycle-G might have a greater effect.

As pointed out by Scholer, the thawing occurs so rapidly on immersion in water at room temperature that the length of cycle would not be increased by elimination of the hot water. Substitution of water at room temperature for the boiling water as suggested by Gonnerman would simplify the procedure and, in the authors' opinion, give substantially the same results. Submersion in hot water was introduced by the authors not so much to hasten the thawing as to attempt to integrate the effect of temperature changes with the effect of freezing and thawing.

# STUDIES OF PAVING CONCRETE

By F. H. Jackson[1] and W. F. Kellermann[2]

## Synopsis

The tests described in this paper were made for the purpose of determining the effect of a number of variables involved in the construction of concrete pavements upon the strength, density and other properties of the pavement slabs. Among the factors studied were:

1. The effect of increasing the percentage of coarse aggregate in the mix.
2. The effect of varying the type of finishing machine.
3. The effect of varying the grading of the coarse aggregate.
4. The effect of varying the consistency of the mix.

The average flexural strength and the uniformity in strength of each pavement slab were obtained by testing large beams taken directly from the test section. These results were compared with the results of tests of beams molded alongside the test section at the time of placing. Compression tests were obtained by drilling cores in each test section and by molding cylinders alongside the test section. The uniformity of the concrete for each test condition was also determined by examining the finished structure for the presence of honeycomb, the amount of honeycomb being used as a measure of the workability of the concrete.

A total of 265 test sections, each 9 ft. wide, 9 ft. long and 7 in. deep, were tested.

As a result of this work, the authors make the following recommendations:

1. That all specifications for concrete for pavements contain a definite requirement covering consistency. The slump test is recommended for this purpose.

2. That the use of paving mixes showing less than a 2-in. slump should be discouraged.

3. That specifications for consistency of paving concrete to be finished by any of the methods now in use be revised when necessary to provide that the concrete have a consistency corresponding to a slump of from 2 to 3 in.

## Introduction

The tests reported in this paper were undertaken primarily for the purpose of determining the effect of variations in the quantity of coarse aggregate upon the strength, density and other properties of concrete pavement slabs which had been placed and finished in accordance with normal field practice. Experience in the use of multiple sizes of coarse aggregate in concrete pavement work in North Carolina and other states during the past few years has indicated the possibility of increasing the quantity of coarse aggregate per unit of volume of concrete beyond the limit which had previously been considered good practice and in this way producing a denser as well as a more economical mixture; provided only that the uniformity of the grading of the coarse aggregate and therefore its void content were controlled rigidly by handling and measuring it in separate sizes.

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[1] Senior Engineer of Tests, U. S. Bureau of Public Roads, Washington, D. C.

[2] Associate Materials Engineer, U. S. Bureau of Public Roads, Washington, D. C.

It was realized at the outset that an investigation of this sort, involving as it did that most elusive property of concrete which we call "workability," could not be performed satisfactorily in the laboratory.

In spite of strenuous efforts on the part of many investigators, no satisfactory laboratory test for workability has as yet been developed. Furthermore, this whole matter of workability is tied up so intimately with the methods of handling and finishing used on the job that it is impossible to set up any laboratory standard which will give more than comparative results. In other words, a concrete which by some laboratory standard may be rated as "unworkable" may be quite workable under certain job conditions. For this investigation the percentage of visible honeycomb in the concrete, as revealed by a careful examination of the slabs, all of which were constructed in accordance with standard field practice, together with the uniformity of strength as determined by testing beams taken directly from the pavement, has been used to measure the uniformity and therefore the workability of the concrete. By this view, any concrete mixture which can be so handled as to produce a uniform homogeneous slab without unduly raising labor costs or reducing efficiency in operation is "workable" concrete, and any concrete which cannot be so handled is not workable, in spite of any rating which it may receive by some arbitrary test.

Most of the information which we have regarding the properties of concrete at the present time has been obtained in the laboratory on small-size beams and cylinders. While these data are of great value in helping us to understand some of the fundamental relationships governing the quality of concrete, as for instance the water-cement ratio - strength law, we must realize that it is the finished structure with which we are primarily concerned and that we are not in a position to make the best use of laboratory test data until we know to just what extent tests on molded specimens measure the quality of the concrete in the structure. This investigation furnished an excellent opportunity to study these relationships, both for crushing strength through the use of molded cylinders compared to cores drilled from the pavement and for transverse strength through the use of molded beams compared to large beams taken from the pavement.

## GENERAL DESCRIPTION OF PROJECT

In order to develop information along these lines, the U. S. Bureau of Public Roads during the summer of 1929 constructed an experimental concrete pavement, 9 ft. in width and approximately $\frac{1}{2}$ mile in length, using standard construction methods and appliances throughout, except that provision was made for creating planes of weakness to permit the removal of the pavement in sections for test purposes. The pavement was built at the Arlington Experimental Station of the Department of Agriculture at Arlington, Va., utilizing for the subgrade the right of way of an abandoned electric line. This provided a graded, well drained, level location which proved to be ideal for the purpose.

A total of 265 sections, each 9 ft. in length, were constructed. The proportions varied from $1:2:3\frac{1}{2}$ by dry rodded volume to $1:2:5\frac{1}{4}$ in the case of gravel, to $1:2:4\frac{3}{4}$ in the case of crushed stone, and to $1:2:4\frac{1}{2}$ in the case of crushed slag. The program called for the construction of six sections per day, three sections of each of two proportions, using a given type and gradation of coarse aggregate and method of finishing. In each group of three, the water content was varied so as to produce a variable consistency ranging from the driest mix which it appeared possible to place without undue effort on the part of the finishers to a consistency approximating a 2 to 3-in. slump. No effort was made to produce wet consistencies such as have sometimes been used in the past because it was felt that the dangers of overwet concrete are sufficiently well known and require no demonstration. On the other hand, the water-cement ratio method of proportioning, the adoption of which is being strongly urged, encourages the use of dry concrete, so that it seemed desirable to study mixtures of this sort rather than the wet consistencies which are recognized as undesirable by everyone.

## MATERIALS AND PROPORTIONS

In order to cover adequately the question of type of coarse aggregate as it affected the workability of the concrete and therefore the limiting quantity which might be used with safety, a siliceous limestone having a rather sharp angular fracture and a bank gravel containing some crushed fragments were used in the tests. In addition, a limited number of sections were laid with blast-furnace slag as coarse aggregate.

The cement was a standard brand of portland cement, meeting all the requirements of the specifications of this Society. It was shipped by car direct from the mill to the site of the work, and all of the cement came from one bin. The test results are shown in Table I.

The fine aggregate was sand from the Potomac River, having a fineness modulus of 2.65. Average test data are given in Table I. It was realized, of course, that the grading and other characteristics of the sand used in these experiments would have a marked effect upon the amount of coarse aggregate which could be used. For instance, it is well known that in general the finer the sand the higher the percentage of coarse aggregate by volume which may be employed without exceeding the limits of workability. However, it was obviously impossible to study any great range in fine aggregate gradations in a program such as this. It was decided, therefore, to use a single

TABLE I.—PHYSICAL PROPERTIES OF CEMENT AND FINE AGGREGATE.

| | |
|---|---|
| **CEMENT**[a] | |
| Fineness, percentage retained on 200-mesh sieve | 14.7 |
| Time of set (Gillmore): Initial | 3 hours, 8 minutes |
| Final | 6 hours, 3 minutes |
| Steam test for soundness | O.K. |
| Normal consistency, per cent | 22.6 |
| Tensile strength, lb. per sq. in. (1:3 Ottawa sand mortar): | |
| 7 days | 315 |
| 28 days | 400 |
| **FINE AGGREGATE** | |
| Sieve analysis:[b] | |
| Total retained on ¼-in. screen, per cent | 1 |
| Total retained on No. 10 sieve, per cent | 14 |
| Total retained on No. 20 sieve, per cent | 32 |
| Total retained on No. 30 sieve, per cent | 45 |
| Total retained on No. 40 sieve, per cent | 69 |
| Total retained on No. 50 sieve, per cent | 86 |
| Total retained on No. 80 sieve, per cent | 94 |
| Total retained on No. 100 sieve, per cent | 96 |
| Total retained on No. 200 sieve, per cent | 97 |
| Silt and clay, per cent | 2.8 |
| Apparent specific gravity | 2.65 |
| Weight, lb. per cu. ft. (dry rodded) | 102 |
| Absorption (Rea's method), per cent | 0.7 |
| Voids, per cent | 38 |
| Organic matter (color test) | O.K. |
| Strength ratio: 7 days | 96 |
| 28 days | 113 |
| Description: Sand consists essentially of angular quartz grains, containing some chert, feldspar and mica. | |

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[a] Results are average of 14 samples tested.

[b] Sieve analysis is average of 96 samples tested.

sand of average gradation with varying percentages of coarse aggregate, beginning at a value sufficiently low to insure a well over-sanded mix for the particular type of sand selected and increasing the amount of coarse aggregate by definite steps until a distinctly under-sanded mix, as judged by laboratory standards, had been obtained.

TABLE II.—PHYSICAL PROPERTIES OF COARSE AGGREGATES.

| Type of Aggregate | | Specific Gravity | Absorption, per cent | Wear, per cent | Weight per Cubic Foot (Dry Rodded), lb. | Voids, per cent |
|---|---|---|---|---|---|---|
| Gravel | Grading A | 2.67 | 0.27 | 14.3 | 108 | 35 |
| | Grading B | 2.64 | 0.43 | | 105 | 36 |
| Stone | Grading A | 2.72 | 0.12 | 4.4[a] | 102 | 40 |
| | Grading B | 2.72 | 0.12 | | 101 | 41 |
| Slag | | 2.47 | .... | 12.2 | 87 | 44 |

[a] Not standard test, made with crushed rock.

The results of the physical tests of the three coarse aggregates are given in Table II. The limestone was from Martinsburg, W. Va.; the gravel from Fredericksburg, Va.; and the slag from Birdsboro, Pa. Each aggregate was ordered shipped to the job in three separate sizes as follows: $\frac{1}{4}$ to $\frac{3}{4}$ in.; $\frac{3}{4}$ to $1\frac{1}{4}$ in.; and $1\frac{1}{4}$ to $2\frac{1}{2}$ in.

The separated sizes were combined by weight to give the combined aggregate gradations indicated in Fig. 1.

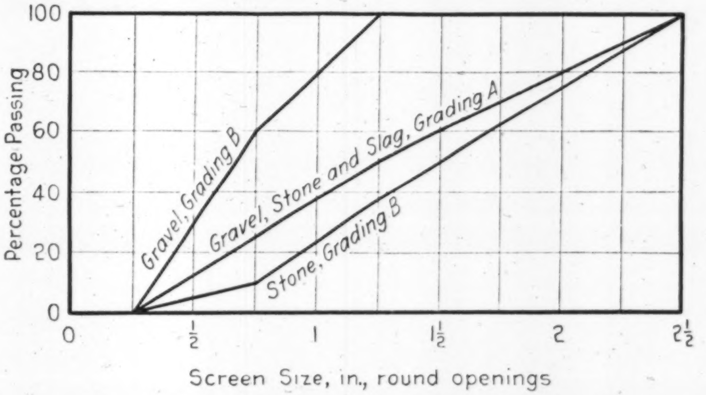

FIG. 1.—Gradings of Coarse Aggregates as Designed.

It will be noted that, in the case of the gravel and the crushed stone, two gradings were employed. Grading A is the so-called straight-line grading, showing an even distribution of sizes from the maximum down to $\frac{1}{4}$ in. Grading B in the case of the gravel ranged in size from $\frac{1}{4}$ to $1\frac{1}{4}$ in., with a surplus in the finer sizes. This grading was used because of the economic importance of this type of gravel in certain sections of the country. Grading B for the crushed stone showed a deficiency in the finer sizes and was used to simulate a condition often met with in practice whenever a demand exists for these finer sizes for bituminous work, resulting in a tendency to rob the concrete aggregate of these sizes.

## CONSTRUCTION METHODS

Standard construction methods and appliances were used in the handling, measurement and hauling of the materials, and in the mixing, placing, finishing and curing of the concrete, except, as has been noted, provision was made for the removal of large slabs from the pavement for test purposes. A detailed description of the procedure is omitted from this paper for lack of space. However, anyone desiring further information may refer to *Public Roads*, the monthly magazine of the U. S. Bureau of Public Roads, the August, 1931, issue of which contains a full report of the investigation.

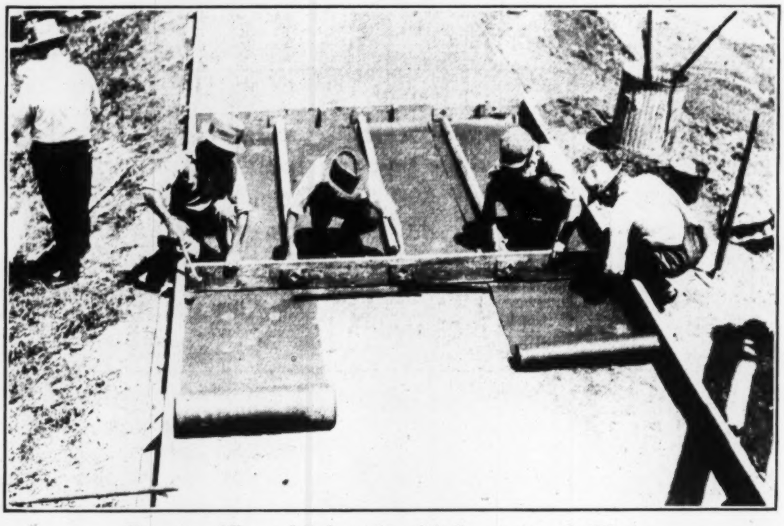

FIG. 2.—View of Subgrade with Separators in Place.

## METHOD OF OBTAINING TEST SLABS

The concrete was laid as a pavement 9 ft. in width, 7 in. in depth, on a flat subgrade which had been very carefully prepared. In order to insure uniform subgrade conditions, single-ply felt tar paper was placed on the subgrade prior to laying the concrete. Transverse headers 9 ft. apart separated the sections, each of which therefore had a volume of 47 cu. ft., or just a little less than the volume of two standard-size batches of a 27E paver. Inasmuch as the concrete was to be actually taken up and tested, three 2 by 2-in. wooden separators were placed between the headers in such a manner as to create longitudinal planes of weakness and divide each 9 by 9-ft. section into four slabs each 27 in. wide by 9 ft. long. These 2 by 2-in. strips were placed at the center of the 7-in. section and, being dry when placed, absorbed sufficient moisture from the concrete to cause swelling with subsequent longitudinal cracking in the concrete. A view of the subgrade showing the separators in place is shown in Fig. 2. Before the concrete had set, a line of $\frac{3}{4}$-in. holes was punched across each section 2 ft. from each end. Just prior to testing, plugs and feathers were inserted in these holes and the 2-ft. end strips broken off. Thus there were obtained from each section four slabs approximately 27 in. wide by 5 ft. long by 7 in. deep.

These slabs were tested in flexure in accordance with the method described later.

## FINISHING

Two finishing machines were employed for finishing the concrete: a combination single screed and tamper, and a double screed machine. In the first round of tests the tamper was used with the single screed machine while in the second round the screed was used without the tamper. The double screed was used in each round of tests. In the tables and charts the single screed with tamper will be referred to as type A, the single screed without tamper as type B, and the double screed as type C. Both machines were equipped with belts so that no hand belting was necessary.

In general, the number of passes of the finishing machine was limited to three, with the idea that this would be about the economic limit from the standpoint of production management on an actual job. However, no more finishing was done in any case than was necessary to secure a satisfactory surface finish. Complete notes were taken giving the number of passes of the finishing machine over each test section, the number of times the concrete was tamped in the case of type A machine and the amount of hand work, if any, necessary.

## CONTROL SPECIMENS

Four 7 by 7 by 30-in. beams and four cylinders were made as control specimens for each test section. These specimens were fabricated in accordance with standard methods and were cured with the pavement slabs until just prior to test. Slump and flow tests for consistency were also made from the concrete for each test section.

Two of the four control beams were tested at 28 days, and the other two at 9 months. The corresponding cylinders were tested at 28 days and 10 months.

## DESCRIPTION OF THE APPARATUS FOR TESTING THE SLABS

Because of the large number of flexure tests to be made on the 27 by 60-in. slabs, it was quite impracticable to make the tests in the laboratory, and field testing was mandatory. Moreover, the importance of the flexure test data was such that accuracy commensurate with that obtained in the laboratory was essential. For these reasons special consideration was given to the items of accuracy and mobility in the design of the apparatus for making these tests.

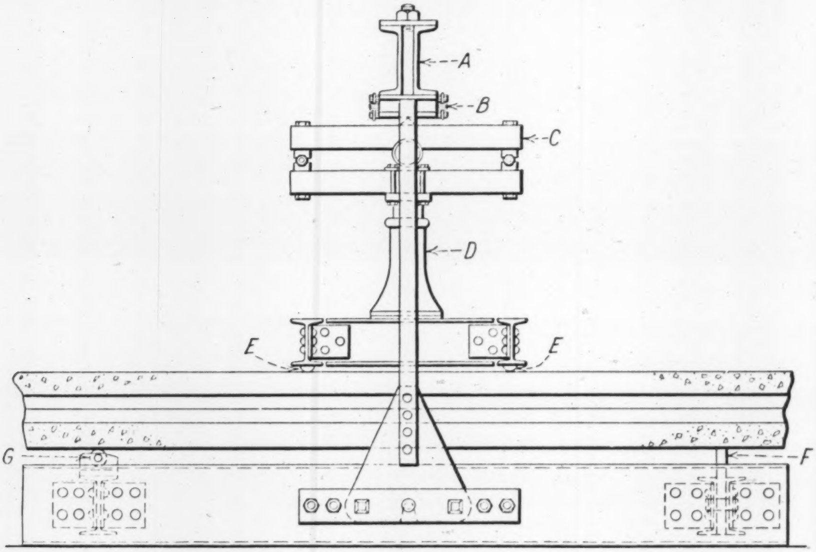

FIG. 3.—Portable Apparatus for Testing Slabs.

The general design was such that the machines could be quickly disassembled into units which could be picked up and moved by hand, yet with a designed load capacity of 20,000 lb., the total load on the specimen could be determined within about 25 lb.

Figure 3 shows a side elevation of one of the testing machines. It consists of a structural steel base frame, which supports a transverse rocker bearing *F* and roller *G*, on which the slab to be tested is supported. At the center of each side member of the base frame, a vertical tension member is attached by two easily removable pins. Passing transversely across the slab at the upper end of these two tension members is a steel beam, *A*, which takes the reaction when a load is applied on the slab. The load is applied by means of a ball-bearing ratchet jack *D* and is distributed to the third points of the span by a structural frame *E*. For determining the magnitude of the applied load a pair of heat-treated steel beams, *C*, is inserted between the head of the jack and the reaction beam *A*. The combined deflection of the two beams is measured with a micrometer dial reading in ten thousandths of an inch. The load deflection characteristics of the beam combination having been determined in advance by calibration, it becomes a simple matter to translate the dial readings into terms of total applied load. This method of load measuring has been used previously[1] and has proved to be very satisfactory. To assure an even bearing of the knife edge located in the center of the upper beam against the reaction, a cylindrical bearing block, *B*, was provided.

In order to take care of the number of tests which had to be made, two complete machines and an extra base frame were built. This permitted the load-applying equipment to be shifted, after the slab failed, to an unbroken specimen set up for test on the extra base, greatly expediting the work.

## TESTING PAVEMENT SLABS

As previously explained, the installation of planes of weakness during construction had caused the formation of three equally-spaced longitudinal cracks in each 9-ft. slab, so that there were available for testing four slabs each 27 in. wide and 9 ft. long. Each of these slabs was reduced in length to 5 ft. just prior to testing by breaking off a section 2 ft. in length from each end. These end sections were discarded entirely because it was felt that, due to the resistance offered by the header boards to free movement of the concrete, the material in the end sections might not be truly representative.

Inasmuch as the test slabs weighed approximately 1000 lb. each, it was necessary to use a special rig for mounting them on the testing machine. A number of *A* frames sufficiently long to span the entire test section and equipped with trolley and chain hoist were employed for this purpose.

When mounting the slabs, a uniform bearing across the entire width at *E*, *F* and *G* was obtained by using plaster of Paris and thin metal strips, except that rubber strips were substituted for plaster of Paris at *E* whenever the top of the slab was smooth enough to permit it.

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[1] *Public Roads*, Vol. 7, No. 2, April, 1926.

The specimens were tested by applying the load by hand as shown in Fig. 4. The magnitude of the load at failure was determined by measuring the deflection of the calibrated beams with the micrometer dial, also shown in the figure.

## CORES DRILLED FROM PAVEMENT

As previously explained, each section was divided into four slabs for testing. These slabs being subsequently broken there resulted eight half-slabs. Of these eight pieces, two were core drilled and the cores tested for absorption at an age of approximately $9\frac{1}{2}$ months, and for crushing strength at an age of 15 months.

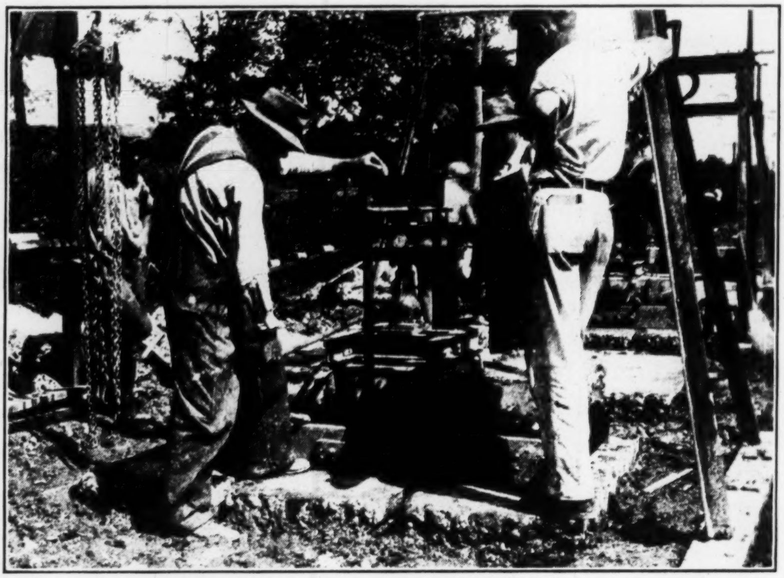

FIG. 4.—Showing a Specimen Being Tested in the Machine.

## DISCUSSION OF TEST RESULTS

The data resulting from these tests are presented both in tabular and in graphic form. The results are shown in detail section by section and are also summarized in various ways in order to bring out certain relationships which appear to have been established.

The discussion will be developed along the following lines: Certain general relationships which appear to exist between the strength and uniformity of the concrete and each of the variables which have been introduced will first be considered. This will be followed by a discussion of the detail results for the purpose of ascertaining to what extent the results for individual sections deviate from the average trends and to determine, if possible, the reasons for such deviations. The following variables will be discussed in the order named, the three coarse aggregates, gravel, crushed stone and slag, being considered separately in each case:

1. Relation between strength of concrete and mix;
2. Relation between strength of concrete and variations in cement factor resulting from change in mix;
3. Relation between strength of concrete and variations in water-cement ratio resulting from change in mix;
4. Effect of type of finishing machine;
5. Effect of grading of coarse aggregate;
6. Effect of honeycomb in slabs on modulus of rupture for each type of finishing machine;
7. Relation between honeycomb in slabs and mix;
8. Relation between honeycomb in slabs and consistency;
9. Uniformity of concrete; and
10. Absorption tests.

### *Relation Between Strength of Concrete and Mix:*

The average results of all tests for each aggregate are plotted in graphic form in Figs. 5 to 7. Each plotted value in these figures represents the average of all tests for the particular mix and type of aggregate indicated. In the case of gravel and stone, each point therefore represents the average of tests on 24 sections except the 1:2:5$\frac{1}{4}$ mix gravel, which represents 14 sections. In the case of slag, each point represents the average of tests on only three sections. Twenty-eight-day and 9-month tests on control specimens for each section are the average of tests on two beams and two cylinders, whereas the tests on the slabs are the average of the four beams taken from the section and the cores the average of tests of two cores drilled from the section. Each point, therefore, with the exceptions noted above, becomes the average of 48 individual tests in the case of the beams, cores and cylinders and the average of 96 individual tests in the case of the pavement slabs.

First, let us examine Fig. 5. A fairly consistent decrease in both flexural and compressive strength is noted for increases in the percentages of coarse aggregate in the mix. It will be observed, however, that, whereas the flexure specimens show about the same percentage of decrease for the slabs at 9 months as for the beams at 28 days and 9 months, the cores tested in compression at 15 months show a somewhat smaller rate of decrease than the cylinders at either 28 days or 10 months. This applies particularly to the leaner mixes. It will be observed also that the percentage of increase in strength from 28 days to 9 months in the case of the beams and from 28 days to 10 months in the case of the cylinders is approximately the same for all mixes, although the average percentage of increase is greater in the case of the compression tests. A very close agreement is shown between the results of beam and slab tests at 9 months for all mixes. The data indicate that in general the results of tests on beam control specimens may be considered to represent very closely the strengths of the slabs. However, these are average relations and it will be shown later during the discussion of the individual test results that such close agreement does not apply under all conditions.

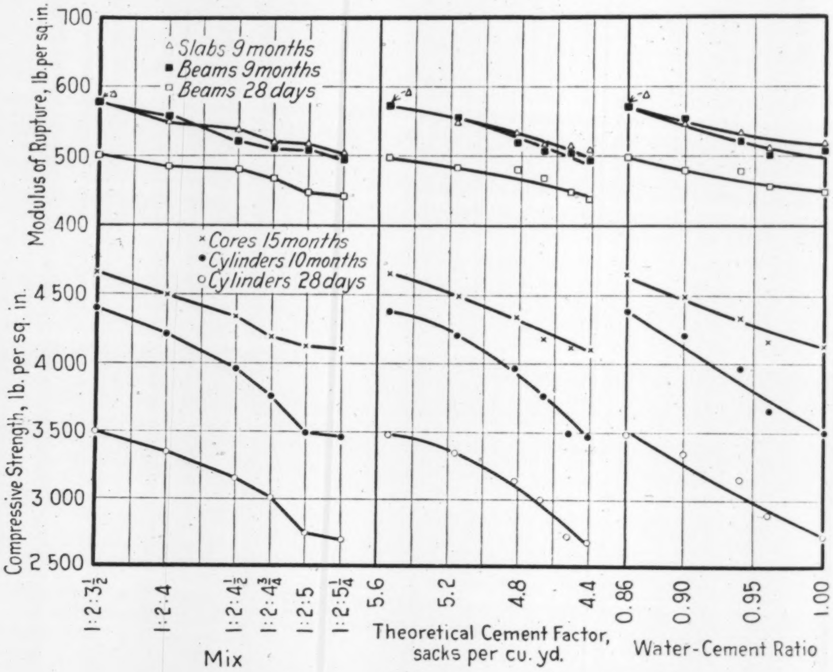

FIG. 5.—Results of Strength Tests of Gravel Concrete.

Unfortunately it was impossible to test the cores at the same age as the 10-month cylinders so an accurate comparison in compression cannot be made. However, the observed increase in cylinder strengths from 28 days to 10 months would indicate, at least in the case of the richer mixtures, a probable further increase to approximately the strengths obtained from the cores at 15 months. It is difficult to predict just what the relation would have been for the mixes containing larger quantities of coarse aggregate, due to the fact that the cylinder strengths decreased at a somewhat greater rate than the core strengths. In general, however, the data from the gravel concrete group seem to indicate that the results of tests on molded cylinders are a fair indication of the crushing strength of the concrete in the pavement.

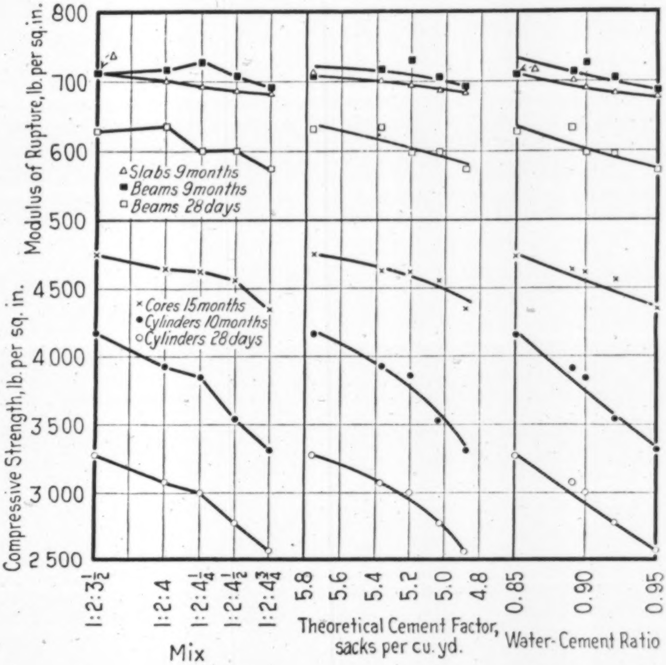

FIG. 6.—Results of Strength Tests of Stone Concrete.

The corresponding values for the crushed stone concrete are shown in Fig. 6. The same general trends are noted as in the case of the gravel concrete. The average reduction in flexural strength for a corresponding increase in coarse aggregate content does not, however, appear to be as great. For instance, in the case of the crushed stone concrete slabs, an increase in coarse aggregate from $3\frac{1}{2}$ to $4\frac{3}{4}$ parts resulted in a decrease in modulus of rupture of only 26 lb. per sq. in., or approximately 4 per cent. The corresponding reduction for the gravel concrete was 52 lb. per sq. in., or 9 per cent. A similar increase in the amount of coarse aggregate reduced the crushing strength of the crushed stone concrete cores 380 lb. per sq. in., or 8 per cent, and the gravel concrete 460 lb. per sq. in., or 10 per cent. On the other hand, there is not the same agreement between results of 9-month beam and slab strengths for the stone concrete as for the gravel concrete. This is particularly true of the 1:2:4¼ mix. In the case of stone concrete the relation between flexural strength and mix is more consistent for the slabs than for the beams, due possibly to the difficulty of molding uniform beam specimens of comparatively small sections (7 by 7 in.) when an angular aggregate is used.

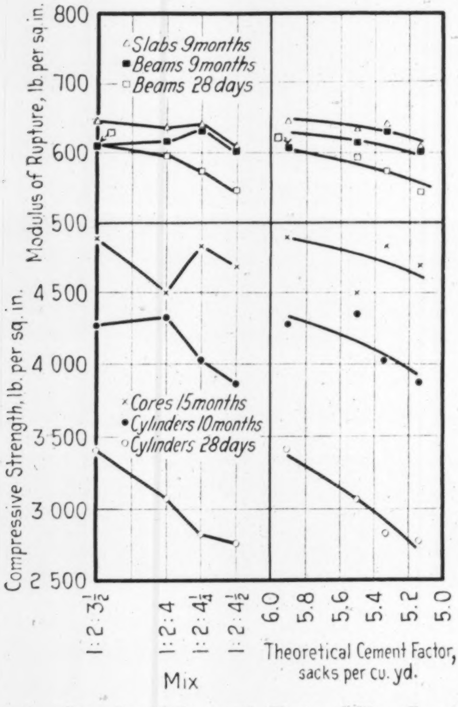

FIG. 7.—Results of Strength Tests of Slag Concrete.

Comparing now the results of compression tests on the stone concrete, as shown in Fig. 6, with the corresponding values on Fig. 5, it will be observed that the difference between cylinder strengths at 10 months and core strengths at 15 months is considerably greater than for the gravel concrete, although like the gravel concrete the rate of decrease with increased percentages of coarse aggregate is greater for the cylinders than for the cores. Inasmuch as the average curing conditions and other factors which might affect the rate of increase in strength were similar, it seems reasonable to assume, in the case of the stone concrete, that the cylinders would not have increased in strength between 10 and 15 months at any greater rate than the gravel concrete. This would indicate the possibility that in the case of the crushed stone concrete, the core strengths at a given age might have been somewhat higher than the control cylinder strengths. However, attention is called to the fact that the difference, if any, would have been relatively small.

Relations between the strength of the slag concrete and mix are shown in Fig. 7. As has been indicated, each value in this case represents the average of tests on three slabs only, the only variable for each proportion being the consistency. Type C finisher only was used with grading A for the slag. In spite of the fact that the results are not so consistent as for the other aggregates, the same general trends may be observed. The most puzzling deviations are the beam tests at 9 months for the 1:2:3½ and 1:2:4 mixes. There appears to be no explanation for these low values, nor for the erratic results in compression for the 1:2:4 mix at 10 months and 15 months. It seems reasonable to assume that these erratic results are purely accidental and that much more consistent values would have been obtained had a larger number of specimens been tested.

### *Relation Between Strength of Concrete and Cement Factor:*

The average flexural and compressive strength results which have just been discussed from the standpoint of mix are also plotted in Figs. 5 to 7 against the cement contents in sacks per cu. yd. of concrete resulting from the arbitrary proportions used. It seemed desirable to study the data from this standpoint because of the difference in cement factor for a given nominal mix due to differences in the voids in the coarse aggregate. Reference to Table II will show that, for the same grading, the gravel contained 5 per cent less voids than the crushed stone and 9 per cent less voids than the crushed slag. These differences are reflected in a variation in yield of concrete to the extent of approximately 0.2 sack per cu. yd. in the case of gravel as compared to crushed stone and about 0.35 sack per cu. yd. in the case of gravel as compared to slag. This is of course due to the fact that the base proportions were by bulk volume. For identical proportions by absolute volume, the yields would have been the same regardless of the difference in void content of the different aggregates.

Certain interesting relationships may be pointed out. For instance, from Figs. 5 and 6 it is seen that, for the flexure specimens and for the cores, the rate of decrease in strength with reductions in cement content is at a fairly uniform rate, whereas in the case of the cylinders, the rate of reduction in the strength of the concrete increased as the cement content is reduced. This applies to both the gravel and stone concrete. For a variation in coarse aggregate from $3\frac{1}{2}$ to $4\frac{3}{4}$ parts, which is the extreme range for the stone concrete, the average decrease in flexural strength, including both stone and gravel concrete, is about 6 per cent. The corresponding average decrease in cement content is approximately 15 per cent. The same increase in coarse aggregate reduced the average crushing strength of the cores about 9 per cent with the same decrease in cement factor, that is, 15 per cent. Attention is called to the fact that the reduction in strength with increasing quantities of coarse aggregate in the mix is, according to the water-cement ratio theory, caused by the dilution of the cement paste, which in turn is brought about by increasing the water content in order to maintain workability. This phase of the matter will be discussed in the next section.

### *Relation Between Strength and Water-Cement Ratio:*

The effect on strength of change in the water-cement ratio brought about by increasing the quantity of coarse aggregate is shown in Figs. 5 and 6. The values for water-cement ratio indicated in these charts are approximate net water-cement ratios after correcting for absorption of the sand but without making any correction for absorption of coarse aggregate. The coarse aggregates were assumed to be in a saturated, surface-dry condition when used and hence no correction should be made in calculating the net water-cement ratio.

It was found impossible to accurately determine the absorption in the slag. Neither does it appear possible to assume that the slag was in a saturated surface-dry condition when used. For these reasons values for net water-cement ratio for the slag concrete were not calculated and no chart is shown for this material.

Examination of Figs. 5 and 6 reveal the conventional water-cement ratio strength relation. It will be observed also that, for a given water-cement ratio, the crushing strengths of the stone concrete cores are approximately the same as the gravel concrete cores, whereas in the case of the control cylinders the gravel concretes show considerably higher crushing strengths than the stone concretes. The difference appears also to become greater as the water-cement ratio increases.

In flexure, the crushed stone concrete for equivalent water-cement ratios is considerably higher in both beam and slab strength, due probably to certain aggregate characteristics inherent in these particular materials.

*Effect of Type of Finishing Machine:*

The average values for modulus of rupture on the slabs and crushing strength on the cores have been plotted by types of finishing machine in Figs. 8 and 9. In these charts each plotted value represents the average of tests on 6 test sections in the case of types A and B finisher and 12 test sections in the case of type C finisher, except for the 1:2:5¼ mix gravel concrete.

According to these curves, the type B finisher gives higher results in flexure than either type A or type C. In compression, on the other hand, type A is high and type B low. These comparisons apply to both types of aggregate although the high values for both flexure and compression are more pronounced in the case of the gravel concrete than for the stone. It will be recalled that the flexure tests were made on the slabs with the load applied on top and with the bottom in tension. It seems quite difficult to explain just why type B finisher should have given values for modulus of rupture higher than either of the other types. This, it will be recalled, was a single screed machine and there seems, at first sight, no reason why it should have compacted the concrete any more thoroughly than the double screed. However, the screed on the type B machine was tilted slightly by raising the front of the screed about ½ in., whereas the screeds on the type C machine were flat. The tilted screed might possibly have a tendency to compact the concrete slightly more than a screed of the type C design.

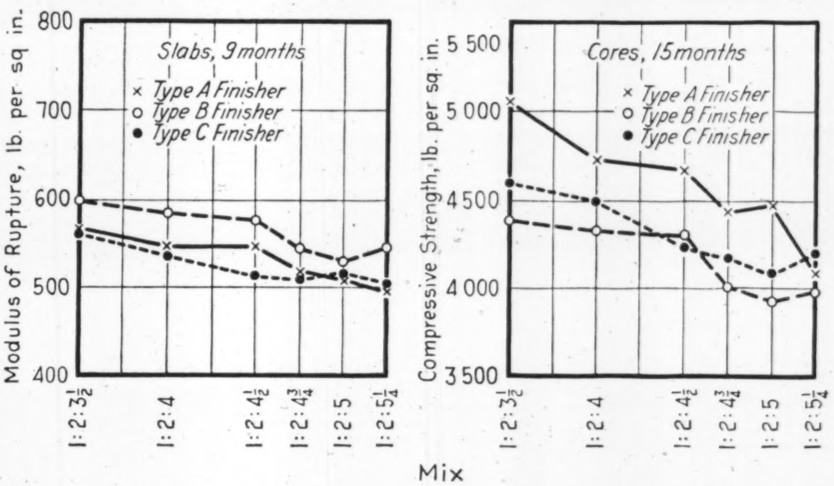

FIG. 8.—Relation Between Strength of Gravel Concrete and Type of Finishing Machine.

In order to determine whether the relatively high modulus of rupture values for type B finisher might have been due to some other factor, as for instance more favorable curing conditions, the average results of the slab tests for each mix, grading, and type finisher have been compared directly with the average results for the corresponding control beams and the "strength ratios," obtained by dividing the average slab strength by the average beam strength and multiplying by 100, used to express the relative efficiency of the different finishers.

These data indicate that, in the case of the gravel concrete and also in the case of the stone concrete, grading A, type B finisher is still high. However, there appears to be no trend in the case of the stone concrete, grading B. Another point of interest is that in the case of the gravel concrete, grading A, there is a tendency for the slabs to show higher strengths in proportion to the beams with increasing amounts of coarse aggregate. This applies to all types of finishers. On the other hand, for the gravel concrete, grading B, the reverse seems to be true except for type B finisher. The tendency for the gravel concrete, grading B, to honeycomb, particularly in the leaner mixes, with consequent falling off in slab strength, probably accounts for this. The effect of honeycombing on strength will be discussed later.

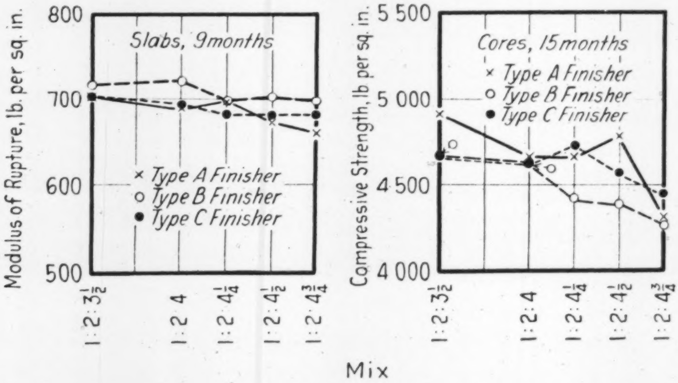

FIG. 9.—Relation Between Strength of Stone Concrete and Type of Finishing Machine.

In the case of the stone concrete there appears to be very little trend as regards the effect of increasing percentages of coarse aggregate except that for grading B the 1:2:3½ mix seems to give relatively high strength ratios for all three types of finishers.

In general it may be said that, aside from a slight advantage in favor of the type B finisher, the analysis of the data indicates that the three types of finishing machines give essentially the same results and also that beam strength is a fairly good indication of the actual flexural strength of the concrete in the structure. In this connection, however, attention should be called to the discussion on the effect of honeycombing in a following section, which shows that this relation does not apply in the case of very dry mixes which honeycomb on placing.

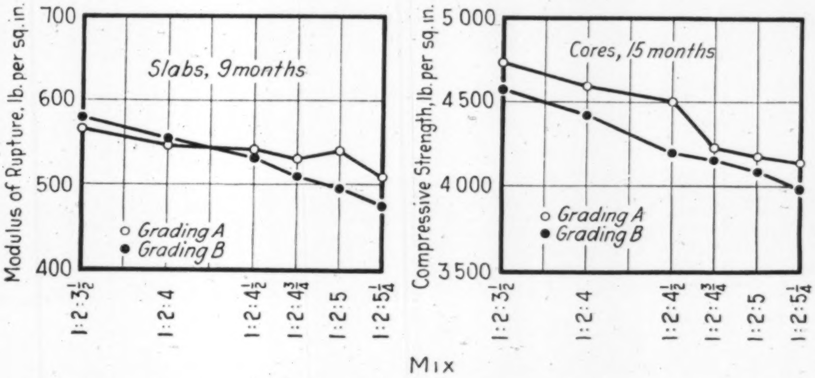

FIG. 10.—Influence of Grading of Coarse Aggregate and Mix upon Strength of Gravel Concrete.

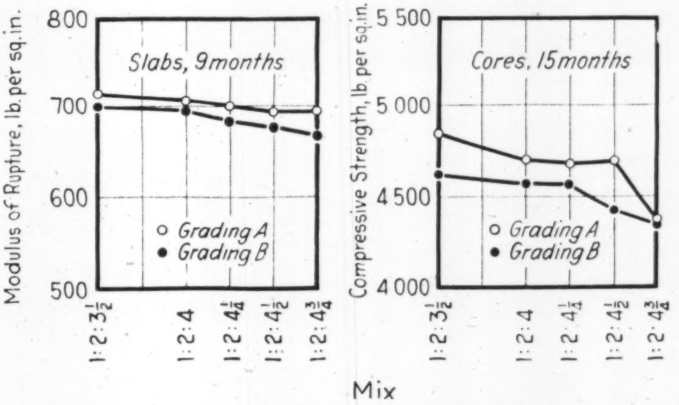

FIG. 11.—Influence of Grading of Coarse Aggregate and Mix upon Strength of Stone Concrete.

*Effect of Grading of Coarse Aggregate:*

The average effect of grading of the coarse aggregate upon the slab and core strengths of the concrete is shown in Figs. 10 and 11. The concrete containing the well-graded coarse aggregate shows

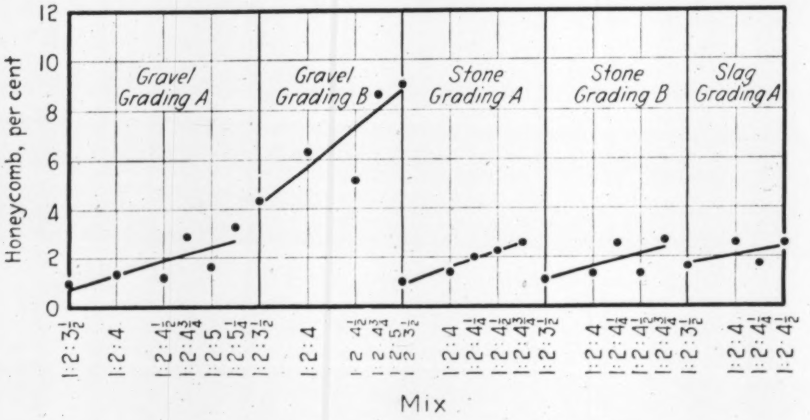

FIG. 12.—Relation Between Honeycomb and Mix.

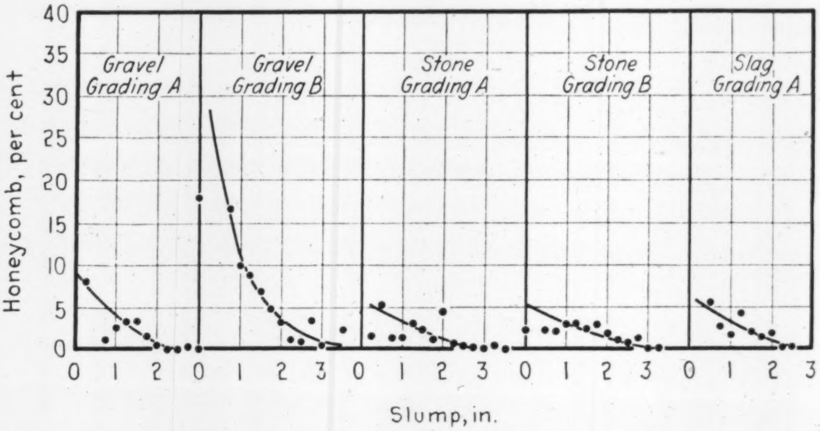

FIG. 13.—Relation Between Honeycomb and Slump.

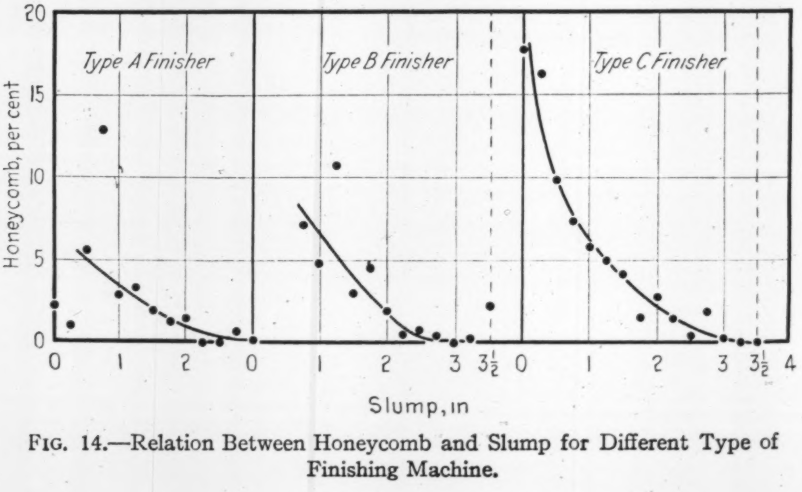

FIG. 14.—Relation Between Honeycomb and Slump for Different Type of Finishing Machine.

somewhat higher strength values than that containing the more poorly graded aggregates although the differences are not very great. For mixes containing more than 4½ parts coarse aggregate, there appears to be a tendency for the concrete containing the small size gravel (grading B) to decrease in flexural strength at a greater rate with increasing amounts of aggregate than in the case of large size material. As has been noted this is no doubt due to the tendency of the poorly graded concrete to honeycomb in the leaner mixes.

*Factors Affecting Honeycomb:*

The percentage of honeycomb in each test slab was determined by averaging two values—one, the percentage of the total area of the bottom of the slab showing honeycomb and, the other, the percentage of the total area of the cross-section at the break which showed honeycomb. It was felt that this method gave a reasonably satisfactory measure of the homogeneity of the concrete in the test slab.

The relationship between average percentages of honeycomb in each mix for each type and gradation of coarse aggregate is shown in Fig. 12 and the corresponding average relationship between honeycomb and consistency is shown in Fig. 13. A similar relation between honeycomb and consistency but with the values classified by types of finisher is shown in Fig. 14. In these figures each point represents the average of the percentages of honeycomb in all the sections having the particular combination of variables involved. In the case of Figs. 13 and 14 the plotted points do not always represent the average of tests on the same number of slabs due to the fact that the same slumps were not always obtained on each of the three sections in each group. In drawing in the average lines, however, each point was weighted in accordance with the number of tests which it represented.

These figures are of interest in showing: (1) that the average amount of honeycomb increases slightly with increases in the percentage of coarse aggregate in the mix (Fig. 12); (2) that where the small size gravel aggregate containing an excess of fine material was used the average amount of honeycomb was considerably increased (Fig. 12); (3) that the average amount of honeycomb was increased by decreasing the slump (Fig. 13); and (4) that honeycombing was somewhat decreased by the use of type A finishing machine (Fig. 14).

One of the most interesting features in connection with the occurrence of honeycomb in the concrete is the effect of grading B in the gravel mixes. Reference to Fig. 1 will show that this material was graded from 1¼ to ¼ in. with 60 per cent passing the ¾-in. screen. The void content was 36 per cent, dry rodded, or 1 per cent greater than grading A. This would seem to be a small difference in voids for so wide a variation in grading. However, repeated tests showed that these values were correct.

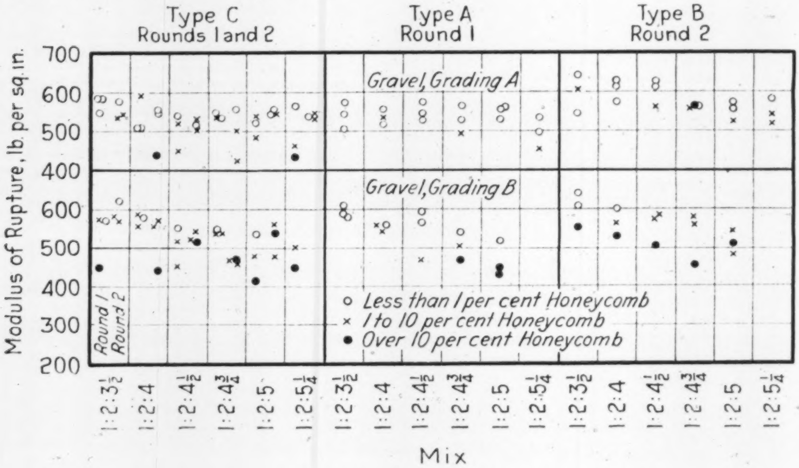

FIG. 15.—Comparison of Modulus of Rupture of Concrete Placed with Different Types of Finishing Machines.

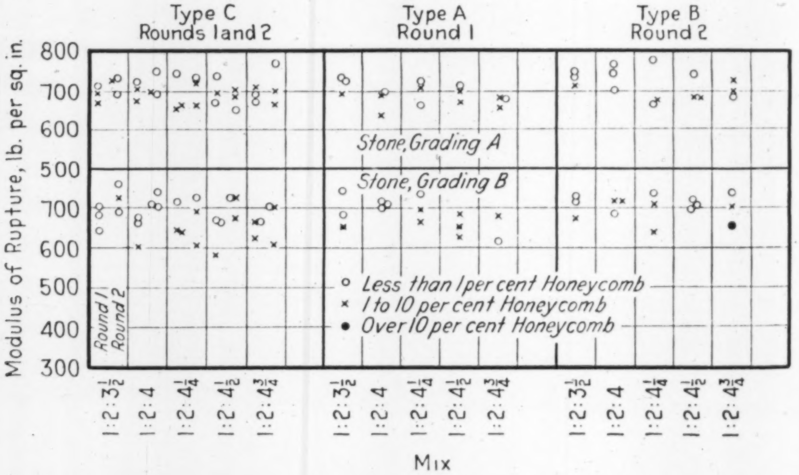

FIG. 16.—Comparison of Modulus of Rupture of Concrete Placed with Different Types of Finishing Machines.

The values for the ratio of volume of mortar to volume of voids in the coarse aggregate and of the ratio of the solid volume of coarse aggregate in a unit volume of concrete to that in a unit volume of coarse aggregate ($b$ to $b_o$) (Talbot and Richart) and for the mortar-voids ratio (volume of mortar, as determined by adding the absolute volumes of cement and sand and the volume of water, divided by the volume of voids in the dry rodded coarse aggregate) were substantially the same for corresponding mixes using gradings A and B. Moreover, the values for these ratios in the 1:2:4 mixes using gravel were just about the same as for the 1:2:3½ mixes using crushed stone. On the assumption that these ratios are significant in controlling workability, this would indicate that the workability of the two gravel concretes should be about the same and also that the 1:2:4 gravel concrete should have about the same workability as the 1:2:3½ stone concrete. Referring, however, to Fig. 12 and assuming that the presence of honeycomb in the finished structure is a reasonably satisfactory measure of the workability of the concrete going into the structure, we find an average of about 1 per cent honeycomb in the 1:2:3½ stone concrete, about 1½ per cent in the 1:2:4 gravel concrete, grading A, and over 6 per cent in the 1:2:4 gravel concrete, grading B. Even the 1:2:3½ concrete, gravel Grading B, showed 4 per cent honeycomb in spite of the fact that this was a distinctly over-sanded mix, when analyzed either by mortar voids or fineness modulus theories. In the latter connection it may be noted that for a 1:2:3½ mix the volume of fine aggregate is 36 per cent of the sum of the volumes of fine and coarse aggregates, which is more sand than would be called for by the fineness modulus theory for these particular aggregates.

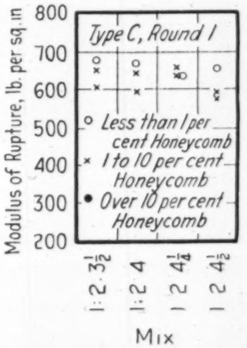

FIG. 17.—Comparison of Modulus of Rupture of Concrete, Slag, Grading A, Placed with Different Types of Finishing Machines.

These tests also indicate that the slump test cannot be used to compare workability of concretes in which different coarse aggregate gradings are used. This is illustrated by reference to Fig. 13. It

TABLE III.—PROPORTIONS AND CONSISTENCY OF CONCRETE.

| Section | Date Laid, 1929 | Proportion | Water-Cement Ratio | Slump, in. | Flow | Theoretical Cement Factor, sacks per cu. yd. | Section | Date Laid, 1929 | Proportions | Water-Cement Ratio | Slump, in. | Flow | Theoretical Cement Factor, sacks per cu. yd. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ROUND 1, GRAVEL AGGREGATE, GRADING A, TYPE A FINISHER | | | | | | | ROUND 1, GRAVEL AGGREGATE, GRADING A, TYPE C FINISHER | | | | | | |
| No. 1 | July 10 | 1:2:3½ | 0.85 | 2 | ... | 5.56 | No. 34 | July 18 | 1:2:3½ | 0.86 | 1 | 123 | 5.54 |
| No. 2 | July 11 | 1:2:3½ | 0.82 | 1¼ | 125 | 5.59 | No. 35 | July 18 | 1:2:3½ | 0.90 | 2½ | 138 | 5.50 |
| No. 3 | July 10 | 1:2:3½ | 0.83 | ¼ | ... | 5.58 | No. 36 | July 18 | 1:2:3½ | 0.87 | 1¾ | 134 | 5.53 |
| No. 4 | July 10 | 1:2:4 | 0.90 | 1¼ | ... | 5.16 | No. 37 | July 18 | 1:2:4 | 0.86 | ¾ | 128 | 5.19 |
| No. 5 | July 10 | 1:2:4 | 0.87 | 1 | ... | 5.18 | No. 38 | July 18 | 1:2:4 | 0.93 | 1 | 123 | 5.12 |
| No. 6 | July 10 | 1:2:4 | 0.93 | 1¾ | ... | 5.12 | No. 39 | July 18 | 1:2:4 | 0.94 | 1¾ | 130 | 5.11 |
| No. 7 | July 11 | 1·2:4½ | 0.90 | 1¼ | 125 | 4.86 | No. 40 | July 19 | 1:2:4½ | 0.91 | 1½ | 120 | 4.85 |
| No. 8 | July 11 | 1:2:4½ | 0.86 | 1 | 122 | 4.89 | No. 41 | July 19 | 1:2:4½ | 0.88 | 1 | 122 | 4.87 |
| No. 9 | July 11 | 1:2:4½ | 0.99 | 2½ | 144 | 4.78 | No. 42 | July 19 | 1:2:4½ | 0.97 | 1¾ | 130 | 4.80 |
| No. 10 | July 11 | 1:2:4¾ | 1.00 | 4 | 155 | 4.64 | No. 43 | July 19 | 1:2:4¾ | 0.90 | 1 | 118 | 4.71 |
| No. 11 | July 11 | 1:2:4¾ | 0.95 | 3 | 150 | 4.68 | No. 44 | July 19 | 1:2:4¾ | 0.98 | 1¾ | 125 | 4.64 |
| No. 12 | July 12 | 1:2:4¾ | 0.87 | 1 | 125 | 4.74 | No. 45 | July 19 | 1:2:4¾ | 1.03 | 1½ | 144 | 4.61 |
| No. 13 | July 12 | 1:2:5 | 0.92 | 1¾ | 130 | 4.57 | No. 46 | July 22 | 1:2:5 | 0.93 | 1¼ | 115 | 4.56 |
| No. 14 | July 12 | 1:2:5 | 1.01 | 5 | 160 | 4.50 | No. 47 | July 22 | 1:2:5 | 1.00 | 2 | 122 | 4.51 |
| No. 15 | July 12 | 1:2:5 | 0.96 | 3 | 148 | 4.53 | No. 48 | July 22 | 1:2:5 | 1.03 | 3 | 144 | 4.49 |
| No. 16 | July 12 | 1:2:5¼ | 0.96 | 2¾ | 138 | 4.42 | No. 49 | July 22 | 1:2:5¼ | 0.93 | 1¾ | 130 | 4.44 |
| No. 17 | July 12 | 1:2:5¼ | 0.91 | 1¾ | 132 | 4.46 | No. 50 | July 22 | 1:2:5¼ | 0.90 | ¼ | 111 | 4.46 |
| No. 18 | July 12 | 1:2:5¼ | 1.03 | 4¾ | 164 | 4.37 | No. 51 | July 22 | 1:2:5¼ | 1.03 | 2½ | 140 | 4.37 |
| ROUND 1, GRAVEL AGGREGATE, GRADING B, TYPE A FINISHER | | | | | | | ROUND 1, GRAVEL AGGREGATE, GRADING B, TYPE C FINISHER | | | | | | |
| No. 19 | July 15 | 1:2:3½ | 0.89 | 1 | 124 | 5.54 | No. 52 | July 23 | 1:2:3½ | 0.84 | ¾ | 125 | 5.60 |
| No. 20 | July 15 | 1:2:3½ | 0.93 | 2¼ | 144 | 5.50 | No. 53 | July 23 | 1:2:3½ | 0.89 | 1½ | 135 | 5.54 |
| No. 21 | July 15 | 1:2:3½ | 0.90 | 1¼ | 127 | 5.53 | No. 54 | July 23 | 1:2:3½ | 0.93 | 2¾ | 143 | 5.50 |
| No. 22 | July 15 | 1:2:4 | 0.85 | 1¼ | 127 | 5.24 | No. 55 | July 23 | 1:2:4 | 0.88 | 1¼ | 127 | 5.21 |
| No. 23 | July 15 | 1:2:4 | 0.92 | 2 | 132 | 5.17 | No. 56 | July 23 | 1:2:4 | 0.95 | 2½ | 140 | 5.14 |
| No. 24 | July 15 | 1:2:4 | 0.96 | 2½ | 136 | 5.14 | No. 57 | July 23 | 1:2:4 | 0.99 | 3 | 142 | 5.10 |
| No. 25 | July 16 | 1:2:4½ | 0.94 | 1¼ | 117 | 4.86 | No. 58 | July 24 | 1:2:4½ | 0.93 | 1½ | 128 | 4.86 |
| No. 26 | July 16 | 1:2:4½ | 1.01 | 1¾ | 138 | 4.80 | No. 59 | July 24 | 1:2:4½ | 0.98 | ¾ | 126 | 4.82 |
| No. 27 | July 16 | 1:2:4½ | 1.05 | 3 | 135 | 4.76 | No. 60 | July 24 | 1:2:4½ | 1.04 | 2 | 142 | 4.77 |
| No. 28 | July 16 | 1:2:4¾ | 1.05 | 2 | 139 | 4.63 | No. 61 | July 24 | 1:2:4¾ | 0.96 | 1¼ | 122 | 4.70 |
| No. 29 | July 16 | 1:2:4¾ | 0.99 | 2 | 139 | 4.68 | No. 62 | July 24 | 1:2:4¾ | 1.03 | 2¼ | 134 | 4.65 |
| No. 30 | July 16 | 1:2:4¾ | 0.96 | 1¼ | 119 | 4.70 | No. 63 | July 24 | 1:2:4¾ | 1.10 | 3 | 146 | 4.59 |
| No. 31 | July 17 | 1:2:5 | 1.04 | 2 | 138 | 4.52 | No. 64 | July 25 | 1:2:5 | 1.02 | ¾ | 122 | 4.53 |
| No. 32 | July 17 | 1:2:5 | 0.92 | ¾ | 120 | 4.61 | No. 65 | July 25 | 1:2:5 | 1.10 | 2 | 132 | 4.47 |
| No. 33 | July 17 | 1:2:5 | 0.96 | 1 | 116 | 4.58 | No. 66 | July 25 | 1:2:5 | 1.13 | 3 | 148 | 4.45 |
| | | | | | | | No. 67 | July 25 | 1:2:5¼ | 1.05 | 2 | 131 | 4.39 |
| | | | | | | | No. 68 | July 25 | 1:2:5¼ | 0.98 | 0 | 115 | 4.44 |

TABLE III.—(*Continued.*)

| Section | Date Laid, 1929 | Proportions | Water-Cement Ratio | Slump, in. | Flow | Theoretical Cement Factor, sacks per cu. yd. | Section | Date Laid, 1929 | Proportions | Water-Cement Ratio | Slump, in. | Flow | Theoretical Cement Factor, sacks per cu. yd. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ROUND 1, STONE AGGREGATE, GRADING A, TYPE A FINISHER | | | | | | | ROUND 1, STONE AGGREGATE, GRADING A, TYPE C FINISHER | | | | | | |
| No. 69 | July 26 | 1:2:3½ | 0.84 | 2½ | 133 | 5.77 | No. 98 | Aug. 2 | 1:2:3½ | 0.83 | 1 | 118 | 5.78 |
| No. 70 | July 26 | 1:2:3½ | 0.81 | 1¼ | 125 | 5.81 | No. 99 | Aug. 2 | 1:2:3½ | 0.88 | 1¾ | 140 | 5.72 |
| No. 71 | July 26 | 1:2:3½ | 0.75 | ¼ | 110 | 5.88 | No. 100 | Aug. 2 | 1:2:3½ | 0.81 | ½ | 118 | 5.81 |
| No. 72 | July 26 | 1:2:4 | 0.84 | 1½ | 127 | 5.42 | No. 101 | Aug. 2 | 1:2:4 | 0.85 | ¾ | 120 | 5.41 |
| No. 73 | July 26 | 1:2:4 | 0.90 | 2¼ | 132 | 5.36 | No. 102 | Aug. 2 | 1:2:4 | 0.90 | 1¾ | 135 | 5.36 |
| No. 74 | July 26 | 1:2:4 | 0.80 | ½ | 112 | 5.47 | No. 103 | Aug. 2 | 1:2:4 | 0.95 | 2½ | 145 | 5.30 |
| No. 75 | July 29 | 1:2:4¼ | 0.88 | ¾ | 120 | 5.22 | No. 104 | Aug. 5 | 1:2:4¼ | 0.87 | 1¼ | 122 | 5.23 |
| No. 76 | July 29 | 1:2:4¼ | 0.94 | 2¼ | 146 | 5.16 | No. 105 | Aug. 5 | 1:2:4¼ | 0.94 | 3 | 144 | 5.16 |
| No. 77 | July 29 | 1:2:4¼ | 0.85 | ¼ | 117 | 5.25 | No. 106 | Aug. 5 | 1:2:4¼ | 0.87 | 1½ | 122 | 5.23 |
| No. 78 | July 29 | 1:2:4½ | 0.85 | ½ | 120 | 5.10 | No. 107 | Aug. 5 | 1:2:4½ | 0.91 | 1½ | 128 | 5.05 |
| No. 79 | July 29 | 1:2:4½ | 0.90 | 1½ | 123 | 5.06 | No. 108 | Aug. 5 | 1:2:4½ | 0.95 | 2¾ | 130 | 5.00 |
| No. 80 | July 29 | 1:2:4½ | 0.94 | 2¼ | 134 | 5.02 | No. 109 | Aug. 5 | 1:2:4½ | 0.99 | 2½ | 142 | 4.97 |
| No. 81 | July 30 | 1:2:4¾ | 0.87 | ½ | 115 | 4.95 | No. 110 | Aug. 6 | 1:2:4¾ | 0.92 | ½ | 112 | 4.90 |
| No. 82 | July 30 | 1:2:4¾ | 0.91 | 1½ | 120 | 4.91 | No. 111 | Aug. 6 | 1:2:4¾ | 0.99 | 1½ | 125 | 4.84 |
| No. 83 | July 30 | 1:2:4¾ | 0.99 | 2 | 133 | 4.84 | No. 112 | Aug. 6 | 1:2:4¾ | 1.04 | 2½ | 150 | 4.80 |
| ROUND 1, STONE AGGREGATE, GRADING B, TYPE A FINISHER | | | | | | | ROUND 1, STONE AGGREGATE, GRADING B, TYPE C FINISHER | | | | | | |
| No. 84 | July 30 | 1:2:3½ | 0.85 | 2 | 138 | 5.81 | No. 113 | Aug. 6 | 1:2:3½ | 0.85 | 1¾ | 127 | 5.81 |
| No. 85 | July 30 | 1:2:3½ | 0.81 | ½ | 116 | 5.86 | No. 114 | Aug. 6 | 1:2:3½ | 0.89 | ½ | 136 | 5.76 |
| No. 86 | July 30 | 1:2:3½ | 0.89 | 2¼ | 132 | 5.76 | No. 115 | Aug. 6 | 1:2:3½ | 0.92 | 3 | 155 | 5.72 |
| No. 87 | July 31 | 1:2:4 | 0.87 | 1 | 124 | 5.43 | No. 116 | Aug. 7 | 1:2:4 | 0.91 | 2 | 140 | 5.39 |
| No. 88 | July 31 | 1:2:4 | 0.91 | 1¾ | 138 | 5.39 | No. 117 | Aug. 7 | 1:2:4 | 0.87 | 1¾ | 144 | 5.43 |
| No. 89 | July 31 | 1:2:4 | 0.94 | 2½ | 146 | 5.36 | No. 118 | Aug. 7 | 1:2:4 | 0.94 | 1¾ | 142 | 5.36 |
| No. 90 | July 31 | 1:2:4¼ | 0.89 | 1½ | 130 | 5.25 | No. 119 | Aug. 7 | 1:2:4¼ | 0.97 | 3 | 156 | 5.17 |
| No. 91 | July 31 | 1:2:4¼ | 0.94 | 2 | 128 | 5.20 | No. 120 | Aug. 7 | 1:2:4¼ | 0.91 | 2 | 146 | 5.23 |
| No. 92 | July 31 | 1:2:4¼ | 0.97 | 2¾ | 144 | 5.17 | No. 121 | Aug. 7 | 1:2:4¼ | 0.85 | 1 | 118 | 5.29 |
| No. 93 | Aug. 1 | 1:2:4½ | 0.89 | 0 | 120 | 5.10 | No. 122 | Aug. 8 | 1:2:4½ | 0.89 | ½ | 117 | 5.10 |
| No. 94 | Aug. 1 | 1:2:4½ | 0.95 | 1¼ | 132 | 5.05 | No. 123 | Aug. 8 | 1:2:4½ | 0.95 | 1½ | 138 | 5.05 |
| No. 95 | Aug. 1 | 1:2:4½ | 0.99 | 1 | 150 | 5.00 | No. 124 | Aug. 8 | 1:2:4½ | 0.98 | 3 | 160 | 5.02 |
| No. 96 | Aug. 1 | 1:2:4¾ | 0.92 | 1¼ | 134 | 4.95 | No. 125 | Aug. 8 | 1:2:4¾ | 0.96 | 2½ | 147 | 4.91 |
| No. 97 | Aug. 1 | 1:2:4¾ | 1.05 | 2 | 158 | 4.83 | No. 126 | Aug. 8 | 1:2:4¾ | 0.94 | 2¾ | 147 | 4.93 |
| | | | | | | | No. 127 | Aug. 8 | 1:2:4¾ | 1.03 | 3 | 180 | 4.85 |

TABLE III.—(*Continued.*)

| Section | Date Laid, 1929 | Propor-tions | Water-Cement Ratio | Slump, in. | Flow | Theoretical Cement Factor, sacks per cu. yd. | Section | Date Laid, 1929 | Propor-tions | Water-Cement Ratio | Slump, in. | Flow | Theoretical Cement Factor, sacks per cu. yd. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ROUND 2, GRAVEL AGGREGATE, GRADING A, TYPE C FINISHER | | | | | | | ROUND 2, GRAVEL AGGREGATE, GRADING A, TYPE B FINISHER | | | | | | |
| No. 128 | Aug. 9 | 1:2:3½ | 0.85 | 1½ | 127 | 5.57 | No. 161 | Aug. 21 | 1:2:3½ | 0.83 | ¾ | 124 | 5.58 |
| No. 129 | Aug. 9 | 1:2:3½ | 0.83 | 1½ | 122 | 5.59 | No. 162 | Aug. 21 | 1:2:3½ | 0.88 | 1 | 120 | 5.52 |
| No. 130 | Aug. 9 | 1:2:3½ | 0.90 | 2 | 130 | 5.51 | No. 163 | Aug. 21 | 1:2:3½ | 0.93 | 2 | 136 | 5.47 |
| No. 131 | Aug. 9 | 1:2:4 | 0.95 | 2½ | 152 | 5.10 | No. 164 | Aug. 21 | 1:2:4 | 0.90 | 1½ | 124 | 5.15 |
| No. 132 | Aug. 9 | 1:2:4 | 0.89 | 2½ | 121 | 5.16 | No. 165 | Aug. 21 | 1:2:4 | 0.95 | 2¼ | 132 | 5.10 |
| No. 133 | Aug. 9 | 1:2:4 | 0.86 | 1 | 124 | 5.19 | No. 166 | Aug. 21 | 1:2:4 | 0.88 | 1 | 127 | 5.17 |
| No. 134 | Aug. 14 | 1:2:4½ | 0.88 | 1½ | 132 | 4.87 | No. 167 | Aug. 22 | 1:2:4½ | 0.88 | 1 | 115 | 4.87 |
| No. 135 | Aug. 14 | 1:2:4½ | 0.98 | 2 | 148 | 4.79 | No. 168 | Aug. 22 | 1:2:4½ | 0.92 | 1½ | 118 | 4.84 |
| No. 136 | Aug. 14 | 1:2:4½ | 0.92 | 2¾ | 147 | 4.84 | No. 169 | Aug. 22 | 1:2:4½ | 0.98 | 1¾ | 124 | 4.79 |
| No. 137 | Aug. 14 | 1:2:4¾ | 0.91 | 1½ | 123 | 4.70 | No. 170 | Aug. 22 | 1:2:4¾ | 0.91 | 1¼ | 123 | 4.70 |
| No. 138 | Aug. 14 | 1:2:4¾ | 0.96 | 2 | 138 | 4.66 | No. 171 | Aug. 22 | 1:2:4¾ | 0.96 | 1¾ | 124 | 4.66 |
| No. 139 | Aug. 14 | 1:2:4¾ | 1.02 | 2½ | 151 | 4.62 | No. 172 | Aug. 22 | 1:2:4¾ | 1.02 | 2 | 133 | 4.62 |
| No. 140 | Aug. 15 | 1:2:5 | 0.93 | 1½ | 118 | 4.56 | No. 173 | Aug. 23 | 1:2:5 | 1.07 | 3 | 152 | 4.46 |
| No. 141 | Aug. 15 | 1:2:5 | 0.99 | 2 | 144 | 4.52 | No. 174 | Aug. 23 | 1:2:5 | 0.99 | 1 | 141 | 4.52 |
| No. 142 | Aug. 15 | 1:2:5 | 1.02 | 2¾ | 130 | 4.49 | No. 175 | Aug. 23 | 1:2:5 | 0.92 | 1 | 124 | 4.57 |
| No. 143 | Aug. 15 | 1:2:5¼ | 0.94 | 1½ | 130 | 4.43 | No. 176 | Aug. 23 | 1:2:5¼ | 1.03 | 2¼ | 138 | 4.37 |
| No. 144 | Aug. 15 | 1:2:5¼ | 0.98 | 1¼ | 119 | 4.40 | No. 177 | Aug. 23 | 1:2:5¼ | 0.98 | 1 | 130 | 4.40 |
| No. 145 | Aug. 15 | 1:2:5¼ | 1.03 | 2¼ | 142 | 4.37 | No. 178 | Aug. 23 | 1:2:5¼ | 0.94 | 1½ | 125 | 4.43 |
| ROUND 2, GRAVEL AGGREGATE, GRADING B, TYPE C FINISHER | | | | | | | ROUND 2, GRAVEL AGGREGATE, GRADING B, TYPE B FINISHER | | | | | | |
| No. 146 | Aug. 16 | 1:2:3½ | 0.84 | 1 | 122 | 5.60 | No. 179 | Aug. 26 | 1:2:3½ | 0.89 | 2 | 133 | 5.54 |
| No. 147 | Aug. 16 | 1:2:3½ | 0.89 | 1½ | 132 | 5.54 | No. 180 | Aug. 26 | 1:2:3½ | 0.85 | 1 | 126 | 5.59 |
| No. 148 | Aug. 16 | 1:2:3½ | 0.94 | 2¾ | 148 | 5.49 | No. 181 | Aug. 26 | 1:2:3½ | 0.94 | 2¼ | 139 | 5.49 |
| No. 149 | Aug. 16 | 1:2:4 | 0.87 | ½ | 117 | 5.22 | No. 182 | Aug. 26 | 1:2:4 | 0.97 | 2½ | 147 | 5.12 |
| No. 150 | Aug. 16 | 1:2:4 | 0.92 | 1¾ | 125 | 5.17 | No. 183 | Aug. 26 | 1:2:4 | 0.92 | 1½ | 133 | 5.17 |
| No. 151 | Aug. 16 | 1:2:4 | 0.97 | 2 | 129 | 5.12 | No. 184 | Aug. 26 | 1:2:4 | 0.87 | ¾ | 113 | 5.22 |
| No. 152 | Aug. 19 | 1:2:4½ | 1.03 | 2 | 133 | 4.78 | No. 185 | Aug. 27 | 1:2:4½ | 1.04 | 3½ | 138 | 4.77 |
| No. 153 | Aug. 19 | 1:2:4½ | 0.98 | 1½ | 123 | 4.83 | No. 186 | Aug. 27 | 1:2:4½ | 0.98 | 1¾ | 137 | 4.82 |
| No. 154 | Aug. 19 | 1:2:4½ | 0.99 | 1 | 124 | 4.81 | No. 187 | Aug. 27 | 1:2:4½ | 0.94 | 1¾ | 124 | 4.86 |
| No. 155 | Aug. 19 | 1:2:4¾ | 0.94 | 1 | 111 | 4.72 | No. 188 | Aug. 27 | 1:2:4¾ | 1.05 | 2 | 145 | 4.63 |
| No. 156 | Aug. 19 | 1:2:4¾ | 1.00 | 2 | 127 | 4.67 | No. 189 | Aug. 27 | 1:2:4¾ | 1.00 | 1¾ | 139 | 4.67 |
| No. 157 | Aug. 19 | 1:2:4¾ | 1.05 | 2¾ | 130 | 4.63 | No. 190 | Aug. 27 | 1:2:4¾ | 0.94 | 1¼ | 125 | 4.72 |
| No. 158 | Aug. 20 | 1:2:5 | 1.01 | 1½ | 115 | 4.54 | No. 191 | Aug. 28 | 1:2:5 | 1.13 | 2½ | 146 | 4.45 |
| No. 159 | Aug 20 | 1:2:5 | 1.07 | 1½ | 129 | 4.49 | No. 192 | Aug. 28 | 1:2:5 | 1.07 | 1 | 163 | 4.49 |
| No. 160 | Aug. 20 | 1:2:5 | 1.13 | 2¾ | 148 | 4.45 | No. 193 | Aug. 28 | 1:2:5 | 1.01 | 1 | 134 | 4.54 |

TABLE III.—(*Continued.*)

| Section | Date Laid, 1929 | Propor-tions | Water-Cement Ratio | Slump, in. | Flow | Theoretical Cement Factor, sacks per cu. yd. | Section | Date Laid, 1929 | Propor-tions | Water-Cement Ratio | Slump, in. | Flow | Theoretical Cement Factor, sacks per cu. yd. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ROUND 2, STONE AGGREGATE, GRADING A, TYPE B FINISHER | | | | | | | ROUND 2, STONE AGGREGATE, GRADING A, TYPE C FINISHER | | | | | | |
| No. 194 | Aug. 29 | 1:2:3½ | 0.88 | 1½ | 134 | 5.72 | No. 224 | Sept. 6 | 1:2:3½ | 0.97 | 4 | 161 | 5.61 |
| No. 195 | Aug. 29 | 1:2:3½ | 0.92 | 2¼ | 156 | 5.57 | No. 225 | Sept. 6 | 1:2:3½ | 0.92 | 2¼ | 156 | 5.67 |
| No. 196 | Aug. 29 | 1:2:3½ | 0.97 | 4 | 155 | 5.61 | No. 226 | Sept. 6 | 1:2:3½ | 0.85 | 1½ | 130 | 5.76 |
| No. 197 | Aug. 29 | 1:2:4 | 0.95 | 2¼ | 146 | 5.30 | No. 227 | Sept. 6 | 1:2:4 | 1.00 | 3¾ | 150 | 5.25 |
| No. 198 | Aug. 29 | 1:2:4 | 0.90 | 1¼ | 131 | 5.36 | No. 228 | Sept. 6 | 1:2:4 | 0.92 | 2¼ | 143 | 5.34 |
| No. 199 | Aug. 29 | 1:2:4 | 1.00 | 3 | 157 | 5.25 | No. 229 | Sept. 6 | 1:2:4 | 0.87 | 1½ | 130 | 5.39 |
| No. 200 | Aug. 30 | 1:2:4¼ | 0.94 | 2½ | 138 | 5.16 | No. 230 | Sept. 9 | 1:2:4¼ | 0.89 | 1¼ | 132 | 5.21 |
| No. 201 | Aug. 30 | 1:2:4¼ | 0.99 | 3 | 157 | 5.11 | No. 231 | Sept. 9 | 1:2:4¼ | 0.94 | 2 | 143 | 5.16 |
| No. 202 | Aug. 30 | 1:2:4¼ | 0.89 | 2 | 128 | 5.21 | No. 232 | Sept. 9 | 1:2:4¼ | 0.99 | 3 | 145 | 5.11 |
| No. 203 | Aug. 30 | 1:2:4½ | 0.99 | 2¾ | 169 | 4.97 | No. 233 | Sept. 9 | 1:2:4½ | 0.92 | 1¼ | 135 | 5.04 |
| No. 204 | Aug. 30 | 1:2:4½ | 0.94 | 2¼ | 147 | 5.02 | No. 234 | Sept. 9 | 1:2:4½ | 0.97 | 2¼ | 133 | 4.99 |
| No. 205 | Aug. 30 | 1:2:4½ | 0.89 | 1¾ | 137 | 5.07 | No. 235 | Sept. 9 | 1:2:4½ | 1.02 | 2½ | 138 | 4.95 |
| No. 206 | Sept. 3 | 1:2:4¾ | 1.04 | 3¼ | 143 | 4.80 | No. 236 | Sept. 10 | 1:2:4¾ | 0.93 | 2 | 131 | 4.89 |
| No. 207 | Sept. 3 | 1:2:4¾ | 0.99 | 2¼ | 146 | 4.84 | No. 237 | Sept. 10 | 1:2:4¾ | 0.99 | 2¼ | 145 | 4.84 |
| No. 208 | Sept. 3 | 1:2:4¾ | 0.93 | 1½ | 131 | 4.89 | No. 238 | Sept. 10 | 1:2:4¾ | 1.04 | 3½ | 166 | 4.80 |
| ROUND 2, STONE AGGREGATE, GRADING B, TYPE B FINISHER | | | | | | | ROUND 2, STONE AGGREGATE, GRADING B, TYPE C FINISHER | | | | | | |
| No. 209 | Sept. 3 | 1:2:3½ | 0.92 | 2¼ | 150 | 5.72 | No. 239 | Sept. 10 | 1:2:3½ | 0.84 | 2 | 142 | 5.82 |
| No. 210 | Sept. 3 | 1:2:3½ | 0.87 | 2 | 135 | 5.78 | No. 240 | Sept. 10 | 1:2:3½ | 0.87 | 2¼ | 147 | 5.78 |
| No. 211 | Sept. 3 | 1:2:3½ | 0.84 | 1½ | 134 | 5.82 | No. 241 | Sept. 10 | 1:2:3½ | 0.92 | 2¾ | 158 | 5.72 |
| No. 212 | Sept. 4 | 1:2:4 | 0.96 | 2 | 144 | 5.34 | No. 242 | Sept. 11 | 1:2:4 | 0.87 | 1½ | 123 | 5.43 |
| No. 213 | Sept. 4 | 1:2:4 | 0.92 | 1¾ | 142 | 5.38 | No. 243 | Sept. 11 | 1:2:4 | 0.92 | 2¼ | 134 | 5.38 |
| No. 214 | Sept. 4 | 1:2:4 | 0.87 | ¾ | 137 | 5.43 | No. 244 | Sept. 11 | 1:2:4 | 0.97 | 1¾ | 154 | 5.33 |
| No. 215 | Sept. 4 | 1:2:4¼ | 1.00 | 2½ | 160 | 5.14 | No. 245 | Sept. 11 | 1:2:4¼ | 0.89 | 1½ | 123 | 5.25 |
| No. 216 | Sept. 4 | 1:2:4¼ | 0.95 | 1¾ | 138 | 5.19 | No. 246 | Sept. 11 | 1:2:4¼ | 0.94 | 2 | 130 | 5.20 |
| No. 217 | Sept. 4 | 1:2:4¼ | 0.90 | 1¾ | 142 | 5.24 | No. 247 | Sept. 11 | 1:2:4¼ | 0.84 | 1 | 120 | 5.30 |
| No. 218 | Sept. 5 | 1:2:4½ | 1.05 | 3¼ | 166 | 4.95 | No. 248 | Sept. 12 | 1:2:4½ | 0.92 | ¾ | 128 | 5.07 |
| No. 219 | Sept. 5 | 1:2:4½ | 0.99 | 2¼ | 162 | 5.01 | No. 249 | Sept. 12 | 1:2:4½ | 0.88 | 2 | 133 | 5.11 |
| No. 220 | Sept. 5 | 1:2:4½ | 0.91 | 2¼ | 142 | 5.08 | No. 250 | Sept. 12 | 1:2:4½ | 0.99 | 3¼ | 145 | 5.01 |
| No. 221 | Sept. 5 | 1:2:4¾ | 1.08 | 4 | 170 | 4.80 | No. 251 | Sept. 12 | 1:2:4¾ | 0.90 | 2¼ | 128 | 4.96 |
| No. 222 | Sept. 5 | 1:2:4¾ | 0.94 | 1¾ | 144 | 4.93 | No. 252 | Sept. 12 | 1:2:4¾ | 0.96 | 2 | 138 | 4.91 |
| No. 223 | Sept. 5 | 1:2:4¾ | 0.90 | 2 | 135 | 4.96 | No. 253 | Sept. 12 | 1:2:4¾ | 0.98 | 2¼ | 145 | 4.89 |

TABLE III.—(*Continued.*)

| Section | Date Laid, 1929 | Proportions | Water-Cement Ratio | Slump, in. | Flow | Theoretical Cement Factor, sacks per cu. yd. | Section | Date Laid, 1929 | Proportions | Water-Cement Ratio | Slump, in. | Flow | Theoretical Cement Factor, sacks per cu. yd. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ROUND 1, SLAG AGGREGATE, GRADING A, TYPE C FINISHER | | | | | | | ROUND 1, SLAG AGGREGATE, GRADING A, TYPE C FINISHER | | | | | | |
| No. 254 | Sept. 13 | $1:2:3\frac{1}{2}$ | 0.87 | $1\frac{1}{2}$ | 126 | 5.91 | No. 260 | Sept. 16 | $1:2:4\frac{1}{4}$ | 0.94 | $1\frac{1}{2}$ | 125 | 5.34 |
| No. 255 | Sept. 13 | $1:2:3\frac{1}{2}$ | 0.92 | $1\frac{3}{4}$ | 128 | 5.84 | No. 261 | Sept. 16 | $1:2:4\frac{1}{4}$ | 0.99 | $1\frac{3}{4}$ | 114 | 5.28 |
| No. 256 | Sept. 13 | $1:2:3\frac{1}{2}$ | 0.83 | $\frac{3}{4}$ | 115 | 5.96 | No. 262 | Sept. 16 | $1:2:4\frac{1}{4}$ | 0.92 | 1 | 118 | 5.36 |
| No. 257 | Sept. 13 | 1:2:4 | 0.87 | $\frac{1}{2}$ | 108 | 5.57 | No. 263 | Sept. 16 | $1:2:4\frac{1}{2}$ | 0.98 | $1\frac{3}{4}$ | 130 | 5.15 |
| No. 258 | Sept. 13 | 1:2:4 | 0.92 | 2 | 124 | 5.51 | No. 264 | Sept. 16 | $1:2:4\frac{1}{2}$ | 1.03 | $2\frac{1}{2}$ | 128 | 5.10 |
| No. 259 | Sept. 13 | 1:2:4 | 0.99 | $2\frac{1}{4}$ | 138 | 5.43 | No. 265 | Sept. 16 | $1:2:4\frac{1}{2}$ | 0.95 | $1\frac{1}{4}$ | 130 | 5.18 |

will be seen that for a given slump, say 1 in., the amount of honeycomb in the case of grading B gravel was much higher than for any other combination.

These data appear also to demonstrate that the use of any function depending directly upon the void content of the coarse aggregate, such as the ratio $b$ to $b_o$ or the mortar voids ratio, does not necessarily control the workability of the mix as determined by the uniformity and freedom from honeycomb of the concrete in the structure. They also indicate that, for a given coarse aggregate type and grading, there is a distinct relation between the consistency of the concrete as measured by the slump test and the percentage of honeycomb apt to develop under the methods of finishing now in common use.

### *Effect of Honeycomb on Flexural Strength of Slabs:*

In Figs. 15 to 17, inclusive, there have been plotted the results of flexure tests on the individual sections arranged by types of finisher and type and gradation of coarse aggregate. Values for strength have been plotted as follows: open circles designating all slabs having less than 1 per cent honeycomb; crosses, all slabs having 1 to 10 per cent honeycomb; solid circles, all slabs having more than 10 per cent honeycomb. The three values in each vertical line represent, of course, tests on the three sections in which the concrete varied only in consistency.

In order to bring out the interrelated effects of honeycomb, strength and consistency, these figures should be considered in connection with the detail results shown in Tables III and IV. Reference to the tables will show a very close relation between honeycomb and consistency in all cases. The concrete in each group of three having the lowest water-cement ratio is almost invariably the concrete having the lowest slump and the largest percentage of honeycomb, if any.

TABLE IV.—RESULTS OF STRENGTH TESTS.

| Section | Proportions | Modulus of Rupture, lb. per sq. in. |  |  | Compressive Strength, lb. per sq. in. |  |  | Honeycomb in Slabs, per cent |
|---|---|---|---|---|---|---|---|---|
|  |  | Beams |  | Slabs | Cylinders |  | Cores |  |
|  |  | 28 days | 9 months | 9 months | 28 days | 10 months | 15 months |  |
| ROUND 1, GRAVEL AGGREGATE, GRADING A, TYPE A FINISHER |  |  |  |  |  |  |  |  |
| No. 1 | 1:2:3½ | 434 | 479 | 542 | 3085 | 3980 | 5320 | 0.1 |
| No. 2 | 1:2:3½ | 468 | 581 | 573 | 2940 | 3900 | 4780 | 0.0 |
| No. 3 | 1:2:3½ | 508 | 582 | 504 | 3300 | 4460 | 4920 | 0.2 |
| No. 4 | 1:2:4 | 427 | 515 | 554 | 2900 | 3940 | 4660 | 0.0 |
| No. 5 | 1:2:4 | 392 | 525 | 532 | 3260 | 4480 | 5120 | 1.3 |
| No. 6 | 1:2:4 | 434 | 471 | 520 | 2475 | 3460 | 3950 | 0.6 |
| No. 7 | 1:2:4½ | 478 | 501 | 529 | 2785 | 3910 | 4700 | 0.0 |
| No. 8 | 1:2:4½ | 489 | 549 | 575 | 2940 | 3950 | 5400 | 0.6 |
| No. 9 | 1:2:4½ | 442 | 491 | 544 | 2410 | 3600 | 4290 | 0.2 |
| No. 10 | 1:2:4¾ | 404 | 453 | 530 | 2250 | 3160 | 4240 | 0.0 |
| No. 11 | 1:2:4¾ | 448 | 490 | 564 | 2495 | 3260 | 4440 | 0.0 |
| No. 12 | 1:2:4¾ | 460 | 434 | 497 | 3145 | 3680 | 4420 | 3.8 |
| No. 13 | 1:2:5 | 452 | 471 | 557 | 3130 | 3880 | 4320 | 0.1 |
| No. 14 | 1:2:5 | 454 | 458 | 529 | 2330 | 3130 | 4270 | 0.0 |
| No. 15 | 1:2:5 | 484 | 455 | 562 | 2690 | 3240 | 4140 | 0.0 |
| No. 16 | 1:2:5¼ | 448 | 409 | 500 | 2340 | 3210 | 3920 | 0.0 |
| No. 17 | 1:2:5¼ | 456 | 526 | 453 | 2975 | 3720 | 4420 | 5.2 |
| No. 18 | 1:2:5¼ | 388 | 460 | 535 | 2100 | 3040 | 3900 | 0.0 |
| ROUND 1, GRAVEL AGGREGATE, GRADING B, TYPE A FINISHER |  |  |  |  |  |  |  |  |
| No. 19 | 1:2:3½ | 446 | 624 | 609 | 3310 | 5000 | 4900 | 0.9 |
| No. 20 | 1:2:3½ | 491 | 661 | 590 | 3515 | 4520 | 5100 | 0.0 |
| No. 21 | 1:2:3½ | 527 | 602 | 580 | 3590 | 4530 | 5260 | 0.0 |
| No. 22 | 1:2:4 | 518 | 614 | 542 | 3580 | 4580 | 5120 | 2.9 |
| No. 23 | 1:2:4 | 556 | 593 | 559 | 3730 | 4670 | 5080 | 3.2 |
| No. 24 | 1:2:4 | 457 | 653 | 561 | 2970 | 4420 | 4360 | 0.0 |
| No. 25 | 1:2:4½ | 510 | 605 | 466 | 3660 | 4450 | 4660 | 6.3 |
| No. 26 | 1:2:4½ | 489 | 585 | 593 | 3200 | 4160 | 4960 | 0.6 |
| No. 27 | 1:2:4½ | 470 | 554 | 566 | 3210 | 3800 | 3940 | 0.6 |
| No. 28 | 1:2:4¾ | 458 | 519 | 541 | 2540 | 3740 | 4660 | 0.2 |
| No. 29 | 1:2:4¾ | 440 | 561 | 503 | 3415 | 4420 | 4150 | 5.8 |
| No. 30 | 1:2:4¾ | 491 | 506 | 468 | 3500 | 4410 | 4620 | 19.2 |
| No. 31 | 1:2:5 | 399 | 478 | 519 | 2470 | 3040 | 4140 | 0.6 |
| No. 32 | 1:2:5 | 469 | 660 | 430 | 3440 | 4750 | 5170 | 25.4 |
| No. 33 | 1:2:5 | 462 | 642 | 447 | 3305 | 4410 | 4770 | 12.9 |
| ROUND 1, GRAVEL AGGREGATE, GRADING A, TYPE C FINISHER |  |  |  |  |  |  |  |  |
| No. 34 | 1:2:3½ | 482 | 662 | 583 | 3360 | 4600 | 5170 | 0.0 |
| No. 35 | 1:2:3½ | 534 | 679 | 547 | 3315 | 4540 | 4820 | 0.0 |
| No. 36 | 1:2:3½ | 544 | 701 | 583 | 3360 | 4310 | 5020 | 0.0 |
| No. 37 | 1:2:4 | 510 | 682 | 594 | 3535 | 4010 | 4880 | 1.2 |
| No. 38 | 1:2:4 | 523 | 670 | 511 | 3165 | 4080 | 4740 | 0.5 |
| No. 39 | 1:2:4 | 494 | 690 | 508 | 3260 | 3920 | 4840 | 0.1 |
| No. 40 | 1:2:4½ | 522 | 500 | 452 | 3280 | 3980 | 4440 | 2.1 |
| No. 41 | 1:2:4½ | 441 | 485 | 518 | 3430 | 4740 | 4450 | 1.0 |
| No. 42 | 1:2:4½ | 426 | 525 | 540 | 2755 | 4040 | 4440 | 0.0 |
| No. 43 | 1:2:4¾ | 441 | 573 | 537 | 3215 | 3600 | 4600 | 4.3 |
| No. 44 | 1:2:4¾ | 448 | 488 | 535 | 3185 | 3820 | 4360 | 0.0 |
| No. 45 | 1:2:4¾ | 461 | 487 | 543 | 2750 | 3700 | 4180 | 0.0 |
| No. 46 | 1:2:5 | 411 | 479 | 537 | 3095 | 3540 | 4140 | 3.9 |
| No. 47 | 1:2:5 | 405 | 516 | 483 | 2570 | 3340 | 4750 | 2.8 |
| No. 48 | 1:2:5 | 396 | 538 | 520 | 2430 | 2800 | 4210 | 0.0 |
| No. 49 | 1:2:5¼ | 393 | 509 | 464 | 2885 | 3370 | 4100 | 6.6 |
| No. 50 | 1:2:5¼ | 386 | 562 | 432 | 3305 | 4380 | 4580 | 16.2 |
| No. 51 | 1:2:5¼ | 397 | 428 | 565 | 2490 | 3470 | 4420 | 0.0 |
| ROUND 1, GRAVEL AGGREGATE, GRADING B, TYPE C FINISHER |  |  |  |  |  |  |  |  |
| No. 52 | 1:2:3½ | 555 | 569 | 447 | 3985 | 4300 | 4310 | 20.5 |
| No. 53 | 1:2:3½ | 588 | 633 | 571 | 3760 | 4280 | 4410 | 8.2 |
| No. 54 | 1:2:3½ | 491 | 553 | 572 | 3480 | 4110 | 4640 | 0.0 |
| No. 55 | 1:2:4 | 427 | 566 | 585 | 3525 | 4190 | 4460 | 7.5 |
| No. 56 | 1:2:4 | 519 | 513 | 555 | 3325 | 3980 | 4430 | 1.2 |
| No. 57 | 1:2:4 | 495 | 545 | 579 | 3270 | 3900 | 4360 | 0.4 |
| No. 58 | 1:2:4½ | 490 | 579 | 451 | 3645 | 4370 | 4230 | 9.8 |
| No. 59 | 1:2:4½ | 503 | 562 | 516 | 3335 | 4280 | 4490 | 4.8 |
| No. 60 | 1:2:4½ | 470 | 511 | 551 | 3145 | 3740 | 4470 | 0.0 |
| No. 61 | 1:2:4¾ | 496 | 599 | 537 | 3715 | 4200 | 4450 | 6.2 |
| No. 62 | 1:2:4¾ | 407 | 530 | 536 | 2970 | 3750 | 4220 | 3.2 |
| No. 63 | 1:2:4¾ | 469 | 495 | 547 | 2785 | 3410 | 3680 | 0.0 |
| No. 64 | 1:2:5 | 371 | 483 | 413 | 3075 | 3810 | 4390 | 24.5 |
| No. 65 | 1:2:5 | 475 | 544 | 477 | 2805 | 3450 | 3850 | 7.0 |
| No. 66 | 1:2:5 | 474 | 590 | 535 | 2645 | 3190 | 3760 | 0.5 |
| No. 67 | 1:2:5¼ | 496 | 524 | 501 | 3110 | 3880 | 3990 | 4.8 |
| No. 68 | 1:2:5¼ | 436 | 593 | 448 | 3175 | 4110 | 3980 | 17.8 |

TABLE IV.—(*Continued.*)

| Section | Proportions | Modulus of Rupture, lb. per sq. in. | | | Compressive Strength, lb. per sq. in. | | | Honeycomb in Slabs, per cent |
|---|---|---|---|---|---|---|---|---|
| | | Beams | | Slabs | Cylinders | | Cores | |
| | | 28 days | 9 months | 9 months | 28 days | 10 months | 15 months | |
| **ROUND 1, STONE AGGREGATE, GRADING A, TYPE A FINISHER** | | | | | | | | |
| No. 69 | 1:2:3½ | 611 | 716 | 732 | 3430 | 4390 | 5410 | 0.0 |
| No. 70 | 1:2:3½ | 614 | 810 | 724 | 3830 | 4600 | 5190 | 0.0 |
| No. 71 | 1:2:3½ | 721 | 822 | 690 | 3990 | 5020 | 5200 | 2.2 |
| No. 72 | 1:2:4 | 702 | 735 | 687 | 3795 | 4430 | 5010 | 1.2 |
| No. 73 | 1:2:4 | 632 | 751 | 697 | 3720 | 4500 | 4860 | 0.0 |
| No. 74 | 1:2:4 | 705 | 793 | 632 | 3890 | 4710 | 5190 | 8.4 |
| No. 75 | 1:2:4¼ | 637 | 791 | 725 | 3380 | 3980 | 4920 | 0.5 |
| No. 76 | 1:2:4¼ | 580 | 788 | 660 | 2685 | 3500 | 4500 | 0.0 |
| No. 77 | 1:2:4¼ | 643 | 860 | 708 | 3575 | 4130 | 5040 | 2.1 |
| No. 78 | 1:2:4½ | 691 | 880 | 670 | 3650 | 4080 | 5340 | 6.5 |
| No. 79 | 1:2:4½ | 647 | 777 | 700 | 3335 | 4140 | 4780 | 2.0 |
| No. 80 | 1:2:4½ | 526 | 738 | 711 | 3020 | 3970 | 4540 | 0.3 |
| No. 81 | 1:2:4¾ | 506 | 754 | 657 | 3070 | 3720 | 4580 | 4.5 |
| No. 82 | 1:2:4¾ | 474 | 786 | 681 | 2725 | 4050 | 4910 | 6.2 |
| No. 83 | 1:2:4¾ | 510 | 752 | 678 | 2420 | 3080 | 4050 | 0.6 |
| **ROUND 1, STONE AGGREGATE, GRADING B, TYPE A FINISHER** | | | | | | | | |
| No. 84 | 1:2:3½ | 541 | 640 | 742 | 3385 | 3930 | 4760 | 0.5 |
| No. 85 | 1:2:3½ | 556 | 713 | 653 | 3480 | 4670 | 4480 | 2.8 |
| No. 86 | 1:2:3½ | 591 | 608 | 683 | 2940 | 3860 | 4400 | 0.0 |
| No. 87 | 1:2:4 | 592 | 724 | 714 | 3445 | 3900 | 4210 | 0.0 |
| No. 88 | 1:2:4 | 571 | 678 | 702 | 2910 | 3770 | 4460 | 0.0 |
| No. 89 | 1:2:4 | 769 | 631 | 711 | 2755 | 3430 | 4200 | 0.0 |
| No. 90 | 1:2:4¼ | 568 | 729 | 695 | 3380 | 4010 | 4760 | 1.6 |
| No. 91 | 1:2:4¼ | 652 | 739 | 732 | 2935 | 3890 | 4520 | 0.8 |
| No. 92 | 1:2:4¼ | 577 | 703 | 666 | 2590 | 3320 | 4180 | 1.4 |
| No. 93 | 1:2:4½ | 590 | 746 | 684 | 2865 | 3840 | 4760 | 2.3 |
| No. 94 | 1:2:4½ | 506 | 769 | 628 | 2655 | 3310 | 4710 | 1.9 |
| No. 95 | 1:2:4½ | 587 | 742 | 654 | 2905 | 3180 | 4410 | 1.0 |
| No. 96 | 1:2:4¾ | 607 | 791 | 678 | 3110 | 3920 | 4620 | 4.2 |
| No. 97 | 1:2:4¾ | 561 | 603 | 613 | 2035 | 3060 | 3330 | 0.4 |

| Section | Proportions | Modulus of Rupture, lb. per sq. in. | | | Compressive Strength, lb. per sq. in. | | | Honeycomb in Slabs, per cent |
|---|---|---|---|---|---|---|---|---|
| | | Beams | | Slabs | Cylinders | | Cores | |
| | | 28 days | 9 months | 9 months | 28 days | 10 months | 15 months | |
| **ROUND 1, STONE AGGREGATE, GRADING A, TYPE C FINISHER** | | | | | | | | |
| No. 98 | 1:2:3½ | 655 | 869 | 670 | 3960 | 4530 | 4960 | 1.4 |
| No. 99 | 1:2:3½ | 679 | 807 | 714 | 3910 | 4380 | 4620 | 0.0 |
| No. 100 | 1:2:3½ | 686 | 865 | 694 | 4180 | 4680 | 4910 | 3.7 |
| No. 101 | 1:2:4 | 686 | 786 | 702 | 3370 | 4510 | 4780 | 2.3 |
| No. 102 | 1:2:4 | 628 | 727 | 725 | 3245 | 4300 | 4600 | 0.0 |
| No. 103 | 1:2:4 | 599 | 762 | 667 | 3565 | 4190 | 4600 | 1.3 |
| No. 104 | 1:2:4¼ | 591 | 772 | 652 | 3720 | 4520 | 4620 | 1.5 |
| No. 105 | 1:2:4¼ | 609 | 739 | 747 | 3810 | 4040 | 4730 | 0.2 |
| No. 106 | 1:2:4¼ | 783 | 749 | 662 | 3420 | 4440 | 4920 | 3.6 |
| No. 107 | 1:2:4½ | 643 | 715 | 694 | 3395 | 4560 | 5490 | 2.0 |
| No. 107 | 1:2:4½ | 695 | 727 | 739 | 3125 | 4110 | 4870 | 0.0 |
| No. 109 | 1:2:4½ | 588 | 727 | 670 | 3055 | 3640 | 4070 | 0.0 |
| No. 110 | 1:2:4¾ | 549 | 641 | 712 | 3330 | 4160 | 4630 | 3.4 |
| No. 111 | 1:2:4¾ | 553 | 698 | 690 | 2940 | 3600 | 4670 | 0.9 |
| No. 112 | 1:2:4¾ | 532 | 577 | 672 | 2090 | 2740 | 4220 | 0.0 |
| **ROUND 1, STONE AGGREGATE, GRADING B, TYPE C FINISHER** | | | | | | | | |
| No. 113 | 1:2:3½ | 569 | 613 | 685 | 3350 | 3850 | 4860 | 0.6 |
| No. 114 | 1:2:3½ | 621 | 654 | 703 | 3010 | 3760 | 4770 | 0.0 |
| No. 115 | 1:2:3½ | 514 | 521 | 644 | 2790 | 3620 | 4550 | 0.0 |
| No. 116 | 1:2:4 | 559 | 719 | 664 | 3045 | 4220 | 4310 | 0.0 |
| No. 117 | 1:2:4 | 608 | 622 | 602 | 3370 | 4180 | 4800 | 9.5 |
| No. 118 | 1:2:4 | 566 | 635 | 676 | 2680 | 3140 | 4640 | 0.0 |
| No. 119 | 1:2:4¼ | 538 | 613 | 713 | 2595 | 2900 | 4410 | 0.0 |
| No. 120 | 1:2:4¼ | 457 | 686 | 645 | 3140 | 3630 | 4620 | 2.6 |
| No. 121 | 1:2:4¼ | 522 | 672 | 640 | 3355 | 4040 | 4720 | 3.8 |
| No. 122 | 1:2:4½ | 559 | 703 | 583 | 2890 | 3460 | 4420 | 3.8 |
| No. 123 | 1:2:4½ | 484 | 682 | 670 | 2330 | 3070 | 4200 | 0.6 |
| No. 124 | 1:2:4½ | 586 | 650 | 666 | 2580 | 2880 | 3880 | 0.0 |
| No. 125 | 1:2:4¾ | 511 | 621 | 625 | 2545 | 3340 | 4100 | 1.7 |
| No. 126 | 1:2:4¾ | 576 | 715 | 666 | 2780 | 3580 | 4500 | 2.2 |
| No. 127 | 1:2:4¾ | 637 | 480 | 666 | 2010 | 2320 | 4030 | 0.0 |

TABLE IV.—(*Continued.*)

| Section | Proportions | Modulus of Rupture, lb. per sq. in., Beams, 28 days | Modulus of Rupture, lb. per sq. in., Beams, 9 months | Modulus of Rupture, lb. per sq. in., Slabs, 9 months | Compressive Strength, lb. per sq. in., Cylinders, 28 days | Compressive Strength, lb. per sq. in., Cylinders, 10 months | Compressive Strength, lb. per sq. in., Cores, 15 months | Honeycomb in Slabs, per cent | Section | Proportions | Modulus of Rupture, lb. per sq. in., Beams, 28 days | Modulus of Rupture, lb. per sq. in., Beams, 9 months | Modulus of Rupture, lb. per sq. in., Slabs, 9 months | Compressive Strength, lb. per sq. in., Cylinders, 28 days | Compressive Strength, lb. per sq. in., Cylinders, 10 months | Compressive Strength, lb. per sq. in., Cores, 15 months | Honeycomb in Slabs, per cent |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ROUND 2, GRAVEL AGGREGATE, GRADING A, TYPE C FINISHER | | | | | | | | | ROUND 2, GRAVEL AGGREGATE, GRADING A, TYPE B FINISHER | | | | | | | | |
| No. 128 | 1:2:3½ | 469 | 502 | 540 | 3415 | 4050 | 4520 | 1.8 | No. 161 | 1:2:3½ | 492 | 550 | 605 | 3900 | 4620 | 4370 | 1.3 |
| No. 129 | 1:2:3½ | 430 | 492 | 536 | 3525 | 4140 | 4380 | 8.1 | No. 162 | 1:2:3½ | 526 | 571 | 641 | 3610 | 4380 | 4480 | 0.0 |
| No. 130 | 1:2:3½ | 475 | 466 | 576 | 3145 | 3920 | 4260 | 0.0 | No. 163 | 1:2:3½ | 533 | 538 | 545 | 3215 | 4030 | 4730 | 0.0 |
| No. 131 | 1:2:4 | 443 | 440 | 543 | 2905 | 3760 | 4350 | 0.0 | No. 164 | 1:2:4 | 488 | 536 | 614 | 3065 | 4320 | 4560 | 0.3 |
| No. 132 | 1:2:4 | 476 | 530 | 556 | 3030 | 3440 | 4270 | 0.0 | No. 165 | 1:2:4 | 538 | 518 | 574 | 3500 | 4030 | 4380 | 0.0 |
| No. 133 | 1:2:4 | 412 | 498 | 440 | 3260 | 3920 | 4760 | 12.0 | No. 166 | 1:2:4 | 532 | 565 | 623 | 3820 | 4600 | 4600 | 0.4 |
| No. 134 | 1:2:4½ | 449 | 493 | 505 | 3150 | 3410 | 4220 | 6.4 | No. 167 | 1:2:4½ | 509 | 494 | 609 | 3150 | 4000 | 4940 | 0.5 |
| No. 135 | 1:2:4½ | 475 | 516 | 512 | 3020 | 3450 | 4220 | 0.0 | No. 168 | 1:2:4½ | 515 | 481 | 558 | 3225 | 4080 | 4520 | 2.8 |
| No. 136 | 1:2:4½ | 463 | 460 | 528 | 3425 | 3890 | 4080 | 1.0 | No. 169 | 1:2:4½ | 466 | 495 | 623 | 2915 | 3740 | 4360 | 0.0 |
| No. 137 | 1:2:4¾ | 490 | 450 | 426 | 3255 | 4000 | 4270 | 8.0 | No. 170 | 1:2:4¾ | 520 | 540 | 558 | 3205 | 3980 | 4060 | 13.7 |
| No. 138 | 1:2:4¾ | 471 | 446 | 501 | 2725 | 3320 | 4160 | 1.6 | No. 171 | 1:2:4¾ | 488 | 516 | 555 | 3220 | 4050 | 3810 | 3.4 |
| No. 139 | 1:2:4¾ | 476 | 470 | 557 | 2280 | 2990 | 4010 | 0.0 | No. 172 | 1:2:4¾ | 496 | 536 | 559 | 2840 | 3700 | 4160 | 0.0 |
| No. 140 | 1:2:5 | 514 | 498 | 544 | 3125 | 3740 | 4420 | 2.9 | No. 173 | 1:2:5 | 465 | 509 | 567 | 2020 | 3000 | 3720 | 0.0 |
| No. 141 | 1:2:5 | 452 | 440 | 547 | 2715 | 3380 | 3920 | 0.0 | No. 174 | 1:2:5 | 478 | 391[a] | 554 | 2265 | 2780 | 3940 | 0.0 |
| No. 142 | 1:2:5 | 519 | 490 | 553 | 2790 | 3100 | 3940 | 0.2 | No. 175 | 1:2:5 | 456 | 443 | 524 | 2860 | 3680 | 4420 | 9.0 |
| No. 143 | 1:2:5¼ | 488 | 468 | 532 | 3020 | 3640 | 4610 | 2.0 | No. 176 | 1:2:5¼ | 439 | 470 | 578 | 2515 | 2980 | 3800 | 0.0 |
| No. 144 | 1:2:5¼ | 452 | 478 | 543 | 2975 | 3470 | 3970 | 2.6 | No. 177 | 1:2:5¼ | 422 | 492 | 514 | 2130 | 2840 | 3980 | 2.8 |
| No. 145 | 1:2:5¼ | 477 | 488 | 535 | 2520 | 3060 | 3860 | 0.3 | No. 178 | 1:2:5¼ | 476 | 508 | 540 | 2325 | 3440 | 4130 | 4.0 |
| ROUND 2, GRAVEL AGGREGATE, GRADING B, TYPE C FINISHER | | | | | | | | | ROUND 2, GRAVEL AGGREGATE, GRADING B, TYPE B FINISHER | | | | | | | | |
| No. 146 | 1:2:3½ | 520 | 649 | 568 | 3915 | 4860 | 4620 | 5.0 | No. 179 | 1:2:3½ | 488 | 597 | 640 | 3460 | 4470 | 4170 | 0.3 |
| No. 147 | 1:2:3½ | 516 | 482 | 579 | 3585 | 4550 | 4370 | 2.9 | No. 180 | 1:2:3½ | 490 | 606 | 552 | 3800 | 4880 | 4480 | 13.5 |
| No. 148 | 1:2:3½ | 519 | 490 | 621 | 3680 | 4460 | 4510 | 0.4 | No. 181 | 1:2:3½ | 446 | 526 | 608 | 3255 | 4240 | 4080 | 0.0 |
| No. 149 | 1:2:4 | 484 | 548 | 439 | 3830 | 4970 | 3950 | 38.2 | No. 182 | 1:2:4 | 470 | 526 | 600 | 3210 | 3980 | 4350 | 0.0 |
| No. 150 | 1:2:4 | 497 | 552 | 568 | 3765 | 4610 | 4450 | 1.4 | No. 183 | 1:2:4 | 461 | 544 | 564 | 3525 | 4460 | 4260 | 1.8 |
| No. 151 | 1:2:4 | 518 | 482 | 556 | 3370 | 4200 | 4400 | 1.6 | No. 184 | 1:2:4 | 530 | 552 | 531 | 4140 | 5040 | 3780 | 17.8 |
| No. 152 | 1:2:4½ | 450 | 502 | 541 | 3065 | 3670 | 3840 | 2.4 | No. 185 | 1:2:4½ | 475 | 516 | 574 | 2780 | 3640 | 3840 | 2.3 |
| No. 153 | 1:2:4½ | 461 | 563 | 512 | 3375 | 4100 | 3830 | 13.0 | No. 186 | 1:2:4½ | 542 | 468 | 584 | 3130 | 4060 | 4010 | 2.9 |
| No. 154 | 1:2:4½ | 475 | 526 | 519 | 3175 | 4060 | 4000 | 4.2 | No. 187 | 1:2:4½ | 503 | 523 | 504 | 3410 | 4200 | 4070 | 14.2 |
| No. 155 | 1:2:4¾ | 472 | 526 | 469 | 3750 | 4260 | 4400 | 24.2 | No. 188 | 1:2:4¾ | 490 | 546 | 580 | 2860 | 3860 | 3760 | 2.0 |
| No. 156 | 1:2:4¾ | 470 | 535 | 464 | 3255 | 4080 | 3960 | 9.6 | No. 189 | 1:2:4¾ | 506 | 494 | 560 | 3105 | 4100 | 4220 | 5.2 |
| No. 157 | 1:2:4¾ | 440 | 482 | 454 | 2765 | 3580 | 3600 | 9.9 | No. 190 | 1:2:4¾ | 474 | 508 | 451 | 3100 | 3490 | 4020 | 18.2 |
| No. 158 | 1:2:5 | 426 | 521 | 540 | 3220 | 3980 | 3840 | 11.3 | No. 191 | 1:2:5 | 432 | 478 | 545 | 2250 | 3240 | 3770 | 1.6 |
| No. 159 | 1:2:5 | 466 | 497 | 476 | 2620 | 3440 | 3940 | 4.4 | No. 192 | 1:2:5 | 452 | 486 | 482 | 2510 | 3540 | 3960 | 7.4 |
| No. 160 | 1:2:5 | 440 | 578 | 561 | 2715 | 3760 | 3720 | 2.5 | No. 193 | 1:2:5 | 399 | 469 | 510 | 2525 | 3710 | 3680 | 10.2 |

[a] One beam only.

TABLE IV.—(*Continued.*)

| Section | Proportions | Modulus of Rupture, lb. per sq. in. | | | Compressive Strength, lb. per sq. in. | | | Honeycomb in Slabs, per cent |
|---|---|---|---|---|---|---|---|---|
| | | Beams | | Slabs | Cylinders | | Cores | |
| | | 28 days | 9 months | 9 months | 28 days | 10 months | 15 months | |
| ROUND 2, STONE AGGREGATE, GRADING A, TYPE B FINISHER | | | | | | | | |
| No. 194 | 1:2:3½ | 772 | 706 | 716 | 3450 | 4820 | 4890 | 2.1 |
| No. 195 | 1:2:3½ | 692 | 710 | 736 | 3540 | 4430 | 4770 | 0.2 |
| No. 196 | 1:2:3½ | 639 | 646 | 748 | 2975 | 3830 | 4750 | 0.0 |
| No. 197 | 1:2:4 | 710 | 700 | 766 | 2865 | 4000 | 4830 | 0.0 |
| No. 198 | 1:2:4 | 662 | 747 | 702 | 3170 | 4340 | 4680 | 0.6 |
| No. 199 | 1:2:4 | 620 | 696 | 743 | 2815 | 3780 | 4810 | 0.0 |
| No. 200 | 1:2:4¼ | 643 | 602 | 665 | 2905 | 3820 | 4650 | 0.8 |
| No. 201 | 1:2:4¼ | 602 | 634 | 778 | 2425 | 3300 | 4370 | 0.0 |
| No. 202 | 1:2:4¼ | 575 | 628 | 676 | 3355 | 4420 | 4760 | 3.9 |
| No. 203 | 1:2:4½ | 632 | 568 | 741 | 2845 | 3660 | 4620 | 0.4 |
| No. 204 | 1:2:4½ | 626 | 590 | 685 | 2910 | 4000 | 4730 | 1.5 |
| No. 205 | 1:2:4½ | 620 | 714 | 684 | 3185 | 3980 | 4860 | 3.3 |
| No. 206 | 1:2:4¾ | 618 | 718 | 683 | 2340 | 3260 | 4140 | 0.4 |
| No. 207 | 1:2:4¾ | 633 | 694 | 731 | 2680 | 3560 | 4280 | 2.8 |
| No. 208 | 1:2:4¾ | 593 | 711 | 702 | 2760 | 3660 | 4380 | 3 0 |
| ROUND 2, STONE AGGREGATE, GRADING B, TYPE B FINISHER | | | | | | | | |
| No. 209 | 1:2:3½ | 604 | 597 | 727 | 2890 | 4110 | 4360 | 0.0 |
| No. 210 | 1:2:3½ | 602 | 676 | 714 | 3000 | 4080 | 4580 | 0.0 |
| No. 211 | 1:2:3½ | 703 | 746 | 670 | 3300 | 4290 | 4740 | 7.8 |
| No. 212 | 1:2:4 | 620 | 749 | 686 | 2750 | 3260 | 4390 | 0.0 |
| No. 213 | 1:2:4 | 593 | 688 | 716 | 2710 | 3460 | 4120 | 1.8 |
| No. 214 | 1:2:4 | 616 | 760 | 716 | 2950 | 3780 | 5000 | 2.6 |
| No. 215 | 1:2:4¼ | 577 | 704 | 736 | 2070 | 2840 | 4080 | 0.0 |
| No. 216 | 1:2:4¼ | 565 | 737 | 708 | 2740 | 3600 | 4350 | 2.2 |
| No. 217 | 1:2:4¼ | 590 | 714 | 638 | 3000 | 4150 | 4260 | 9.6 |
| No. 218 | 1:2:4½ | 576 | 621 | 696 | 1830 | 2830 | 3660 | 0.0 |
| No. 219 | 1:2:4½ | 566 | 592 | 716 | 2020 | 2560 | 4200 | 0.0 |
| No. 220 | 1:2:4½ | 609 | 772 | 703 | 2700 | 3570 | 4200 | 0.5 |
| No. 221 | 1:2:4¾ | 565 | 700 | 734 | 1590 | 2040 | 3670 | 0.0 |
| No. 222 | 1:2:4¾ | 622 | 756 | 701 | 2410 | 3000 | 4400 | 3.0 |
| No. 223 | 1:2:4¾ | 632 | 750 | 651 | 2900 | 3400 | 4700 | 10.2 |
| ROUND 2, STONE AGGREGATE, GRADING A, TYPE C FINISHER | | | | | | | | |
| No. 224 | 1:2:3½ | 603 | 680 | 691 | 2750 | 3500 | 4420 | 0.0 |
| No. 225 | 1:2:3½ | 567 | 735 | 736 | 2890 | 3940 | 4460 | 0.0 |
| No. 226 | 1:2:3½ | 603 | 686 | 730 | 2930 | 3820 | 4660 | 1.8 |
| No. 227 | 1:2:4 | 646 | 706 | 752 | 2700 | 3320 | 4180 | 0.0 |
| No. 228 | 1:2:4 | 598 | 682 | 691 | 2870 | 3640 | 4420 | 0.0 |
| No. 229 | 1:2:4 | 593 | 684 | 700 | 2920 | 4380 | 4450 | 3.6 |
| No. 230 | 1:2:4¼ | 584 | 812 | 662 | 3220 | 4410 | 4800 | 4.7 |
| No. 231 | 1:2:4¼ | 588 | 790 | 723 | 2900 | 4060 | 4410 | 6.2 |
| No. 232 | 1:2:4¼ | 543 | 772 | 737 | 2480 | 3760 | 4500 | 0.0 |
| No. 233 | 1:2:4½ | 614 | 651 | 687 | 2980 | 4140 | 4580 | 8.8 |
| No. 234 | 1:2:4½ | 586 | 812 | 701 | 2690 | 3820 | 4300 | 2.2 |
| No. 235 | 1:2:4½ | 504 | 728 | 650 | 2430 | 3360 | 4200 | 0.1 |
| No. 236 | 1:2:4¾ | 556 | 656 | 700 | 2880 | 4020 | 4290 | 7.9 |
| No. 237 | 1:2:4¾ | 590 | 716 | 665 | 2510 | 3720 | 4320 | 1.1 |
| No. 238 | 1:2:4¾ | 557 | 626 | 772 | 2140 | 2930 | 3940 | 0.0 |
| ROUND 2, STONE AGGREGATE, GRADING B, TYPE C FINISHER | | | | | | | | |
| No. 239 | 1:2:3½ | 660 | 802 | 726 | 3170 | 4260 | 4690 | 1.9 |
| No. 240 | 1:2:3½ | 577 | 684 | 761 | 2850 | 4020 | 4320 | 0.0 |
| No. 241 | 1:2:3½ | 662 | 631 | 690 | 2540 | 3520 | 4840 | 0.0 |
| No. 242 | 1:2:4 | 636 | 772 | 739 | 3060 | 3860 | 5150 | 0.4 |
| No. 243 | 1:2:4 | 608 | 678 | 702 | 2700 | 3770 | 4880 | 0.8 |
| No. 244 | 1:2:4 | 623 | 686 | 708 | 2430 | 3260 | 4720 | 0.0 |
| No. 245 | 1:2:4¼ | 629 | 698 | 691 | 2720 | 3840 | 5180 | 1.4 |
| No. 246 | 1:2:4¼ | 578 | 704 | 726 | 2190 | 3570 | 4760 | 0.0 |
| No. 247 | 1:2:4¼ | 665 | 762 | 605 | 3290 | 4440 | 5000 | 7.4 |
| No. 248 | 1:2:4½ | 664 | 696 | 726 | 2350 | 3080 | 4900 | 1.6 |
| No. 249 | 1:2:4½ | 656 | 711 | 675 | 2730 | 3200 | 5080 | 4.2 |
| No. 250 | 1:2:4½ | 553 | 596 | 724 | 2090 | 2770 | 4690 | 0.0 |
| No. 251 | 1:2:4¾ | 610 | 698 | 611 | 2770 | 3120 | 4930 | 5.8 |
| No. 252 | 1:2:4¾ | 554 | 675 | 700 | 2370 | 3120 | 4760 | 1.8 |
| No. 253 | 1:2:4¾ | 565 | 734 | 701 | 2360 | 2920 | 4760 | 0.4 |

TABLE IV.—(*Continued.*)

| Section | Proportions | Modulus of Rupture, lb. per sq. in. | | | Compressive Strength, lb. per sq. in. | | | Honeycomb in Slabs, per cent |
|---|---|---|---|---|---|---|---|---|
| | | Beams | | Slabs | Cylinders | | Cores | |
| | | 28 days | 9 months | 9 months | 28 days | 10 months | 15 months | |
| ROUND 1, SLAG AGGREGATE, GRADING A, TYPE C FINISHER | | | | | | | | |
| No. 254 | 1:2:3½ | 623 | 618 | 650 | 3160 | 4290 | 4860 | 1.4 |
| No. 255 | 1:2:3½ | 598 | 620 | 679 | 3300 | 4040 | 4940 | 0.6 |
| No. 256 | 1:2:3½ | 604 | 584 | 603 | 3740 | 4500 | 4890 | 2.8 |
| No. 257 | 1:2:4 | 598 | 586 | 642 | 3480 | 4660 | 4880 | 5.5 |
| No. 258 | 1:2:4 | 579 | 630 | 592 | 3080 | 4220 | 4190 | 2.0 |
| No. 259 | 1:2:4 | 603 | 623 | 668 | 2620 | 4150 | 4440 | 0.2 |
| ROUND 1, SLAG AGGREGATE, GRADING A, TYPE C FINISHER | | | | | | | | |
| No. 260 | 1:2:4¼ | 553 | 616 | 635 | 2830 | 3860 | 4660 | 2.8 |
| No. 261 | 1:2:4¼ | 551 | 624 | 636 | 2530 | 3760 | 4760 | 0.6 |
| No. 262 | 1:2:4¼ | 618 | 646 | 650 | 3130 | 4460 | 5110 | 1.8 |
| No. 263 | 1:2:4½ | 570 | 593 | 592 | 2770 | 3780 | 4660 | 3.2 |
| No. 264 | 1:2:4½ | 516 | 580 | 653 | 2600 | 3520 | 4760 | 0.3 |
| No. 265 | 1:2:4½ | 547 | 620 | 576 | 2950 | 4300 | 4710 | 4.4 |

Referring again to the charts showing relationship between honeycomb and strength (Figs. 15 to 17, inclusive), the data indicate conclusively the danger of permitting any condition which will tend to the formation of honeycomb. In the whole group (265 sections) there are 19 sections having more than 10 per cent honeycomb, 15 of which were in the gravel grading B group. In all but four cases the badly honeycombed section was distinctly lower in strength than the other sections of the same group. The same relation exists for the slabs showing from 1 to 10 per cent honeycomb. There were 119 such sections. In only 26 of these was the strength of the honeycombed slab as high or higher than any section in the group showing less than 1 per cent honeycomb. These results emphasize again the danger of using very dry concrete with our present methods of finishing.

A number of interesting detail comparisons may be made by reference to Figs. 15 to 17. For instance, it will be noted that there is almost a complete absence of honeycomb in the sections containing gravel, grading A, and finished with type A machine (Fig. 15). The corresponding group using stone has a considerably larger number of slabs showing from 1 to 10 per cent honeycomb. Concrete, gravel, grading B, also showed somewhat less honeycombing when finished with type A machine than with either of the other types, although, as has previously been pointed out, the amount of honeycombing in general was much greater for grading B than for grading A (Fig. 15). The type of finisher appeared to make little difference in the case of the stone concrete, sections finished with each type showing about the same amount of honeycomb.

*Uniformity of Concrete:*

It will be recalled that the value for the modulus of rupture of each test slab was obtained by averaging the results of tests on the four slabs which made up the section. A study of the variation in strength among the four slabs should give some idea, therefore, of the uniformity of the concrete within the section.

In Fig. 18 the average percentage variations in strength for all of the gravel concrete sections and all of the stone concrete sections for each proportion have been plotted. It will be seen that there is a reasonably consistent reduction in uniformity with increase in coarse aggregate content for both types of coarse aggregate. The average deviation varies from about 4 per cent to about 7 per cent for the entire range. By breaking these average values up into group averages corresponding to grading of aggregate, types of finisher, etc., similar results are obtained. The difference between the effect of gradings A and B in the gravel which is so marked as regards honeycombing does not appear from the standpoint of uniformity. Neither can any consistent difference in uniformity be observed as the result of using the different methods of finishing. Each method gives about the same average degree of uniformity.

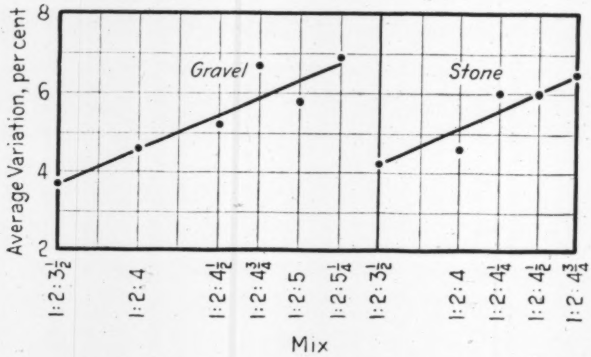

FIG. 18.—Uniformity of Concrete. Average percentage variation of modulus of rupture of individual slabs from modulus of rupture for the section (4 slabs).

*Absorption:*

In order to develop information which might be of assistance in studying the relative durability of the various mixes, a series of absorption tests was made on 6-in. cores drilled from the test sections. These tests were made in accordance with the standard 5-hour immersion in the boiling water method as specified by the Society.[1] The average percentage by weight of water absorbed for each proportion and for each type of finishing machine is given in Fig. 19. The average absorption is about 5.5 per cent for all conditions with the exception of the slag concrete, which averages about 7.5 per cent, and the stone concrete finished with type A finisher, which shows about 5 per cent. There is a slight tendency for the absorption to decrease with increasing quantities of coarse aggregate.

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[1] See the Absorption Test Procedure described in the Tentative Methods of Testing Brick (C 67 – 30 T), *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part I, p. 1036 (1930;) also 1930 Book of A.S.T.M. Tentative Standards, p. 201.

These tests are being supplemented now with a series of freezing-and-thawing tests on a similar series of cores. It is hoped that it will be possible to present a report covering this portion of the investigation at an early date.

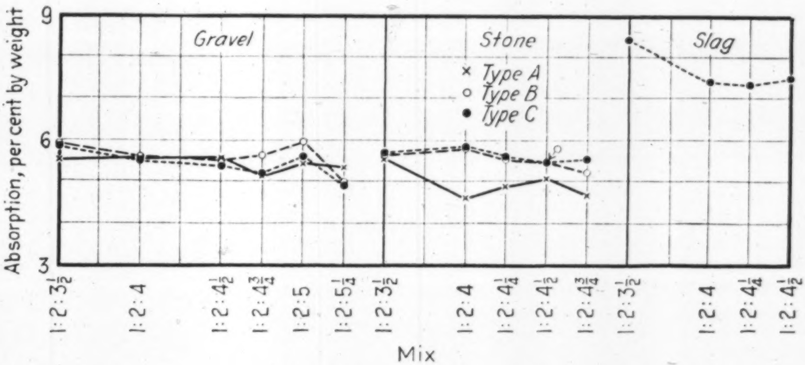

FIG. 19.—Relation Between Absorption of Cores at Nine Months and Type of Finishing Machine.

## STUDIES OF INDIVIDUAL TEST SECTIONS

In order to make it possible to study the manner in which the several variables which were being investigated affected the relative strength of individual sections, it was considered advisable to plot the results of tests section by section. This was done in a series of charts which appear in the complete report and in which the results of the various strength tests on each section have been indicated together with the mix, slump, water-cement ratio, and average percentages of honeycomb. The individual test data from which these graphs were prepared are shown in Tables III and IV. There are several points in connection with the tests on the individual sections to which attention is especially called:

1. For each group of tests a very distinct relation exists between the consistency of the concrete as measured by the slump test and the water-cement ratio. Almost invariably it will be found that, in each group of three sections, a direct relation exists which indicates

that the slump test is a very good measure of the *relative* amount of water in a given concrete, even within the rather narrow limits covered by this study.

2. In general, the additional water required to maintain workability in the mixtures containing high percentages of coarse aggregate does not result in an increase in slump.

3. The crushing strength of the concrete in the individual sections follows, in general, the water-cement ratio law. Exceptions may be observed in certain instances, many of which may be explained by the use of concrete which is too dry to be workable. It must be remembered, of course, that each value represents the average of only two tests in the case of the control specimens and the cores and four tests in the case of the slabs, so too great concordance should not be expected.

4. There is a marked tendency for the flexural strength of both beams and slabs to fall off for the dry mixes. The tendency is more marked in the case of the slabs, due probably to the fact that, under the standard method of molding employed for the control beams, it was possible to fabricate more uniform specimens with the drier mixes than by the methods employed in placing the slabs. This tendency is illustrated by reference to Sections Nos. 3, 17, 25, 32, 50, 71, 85, 122, 149, 180, 211, 230 and 247.

5. In 60 out of a total of 89 groups of three sections each, the section having the highest water-cement ratio showed a slab strength as high or higher than the corresponding section in the same group having the lowest water-cement ratio. Attention is called to the fact that, in each group of three sections, the range in water-cement ratio was only sufficient to produce a variation in consistency from dry to medium and that in no case was the water-cement ratio high enough to produce concrete of wet or sloppy consistency.

It should also be noted that, whereas the drier mixes would for the most part be rated as workable by laboratory standards, such mixes were not in general found to be workable under the methods of placing and finishing pavement slabs now commonly used. The dry mixes may therefore be considered as lying outside of the range of plastic mixes to which the water-cement ratio law applies, which accounts for the falling off in strength.

6. In these same 60 groups, the section in each group having the lowest water-cement ratio was honeycombed in all cases. On the other hand, in 40 out of the 60 groups, honeycomb was entirely absent from the section having the highest water-cement ratio with 13 out of the remaining 20 showing less than 1 per cent honeycomb.

7. The slump for the driest mix in each group averaged from $\frac{1}{4}$ to 1 in., while the slump for the corresponding section in each group having the highest water-cement ratio averaged from 2 to 3 in.

The above indications point emphatically to the danger of using mixes of less than a 2-in. slump in concrete paving work with the methods of finishing in common use today. The use of medium consistencies such as were obtained in these tests with a 2 to 3-in. slump will not only give as high or higher slab strength than the drier mixes but will greatly decrease the tendency to honeycomb and thus will promote uniformity.

8. In general, the presence of honeycomb in the slabs, although very seriously affecting the flexural strengths of the slabs themselves, was not accompanied by lower crushing strengths of the cores drilled from the slabs. Neither was there any relation between the extent of honeycomb in the test slabs and the amount of honeycomb in the cores drilled therefrom. These observations lead to the conclusion that neither the crushing strength or freedom from honeycomb of cores is necessarily an indication of the homogeneity of the concrete in the slab.

## CONCLUSIONS

For the materials, proportions and methods of placing used in these tests the results of the investigation justify the following conclusions:

1. For a constant sand-cement ratio, an increase in the quantity of coarse aggregate beyond the limits ordinarily employed in practice decreased the strength of the concrete in the pavement substantially in proportion to the amount of additional water required to maintain workability.

2. For corresponding increases in the water-cement ratio the percentage of reduction in flexural strength was somewhat less than the percentage of reduction in crushing strength.

3. The workability and uniformity in strength of the concrete was decreased in proportion to the amount of coarse aggregate added.

4. The workability of the concrete was greatly reduced by decreasing the maximum size and at the same time increasing the amount of fines in the gravel coarse aggregate, even though the corresponding variations in the percentage of voids in the gravel and the corresponding variations in the ratio of $b$ to $b_o$ for the concrete was small.

5. For the gravel concrete the reduction in workability for any mix due to the use of the poorly graded gravel was greater than the reduction in workability due to increasing the percentage of coarse

aggregate in the concrete containing the well-graded material (see Fig. 12).

6. For a given consistency as measured by the slump test, different gradations of gravel produced different degrees of workability (Fig. 13).

7. For a given gradation of coarse aggregate the consistency as determined by the slump test measured the workability of the concrete.

8. For a given proportion, higher slab strengths were obtained with concrete having a medium consistency (2 to 3-in. slump) than with very dry concrete ($\frac{1}{4}$ to 1-in. slump).

9. In order to secure satisfactory uniformity and freedom from honeycomb it was necessary to use a consistency corresponding to a slump of approximately 2 to 3 in.

10. The various types of finishing machines employed in these tests gave approximately the same results.

11. For concrete reasonably free from honeycomb, the strength of the control beams appeared to be a satisfactory measure of the strength of the corresponding pavement slab.

12. Neither the amount of honeycombing observed in the drilled cores or the crushing strengths developed by the drilled cores measured the extent of honeycomb in the pavement slabs or the flexural strength developed by the pavement slabs.

*Recommendations:*

(*a*) That all specifications for concrete for pavements contain a definite requirement covering consistency. The slump test is recommended for this purpose.

(*b*) That the use of paving mixes showing less than a 2-in. slump should be discouraged.

(*c*) That specifications for consistency of paving concrete to be finished by any of the methods now in use be revised when necessary to provide that the concrete have a consistency corresponding to a slump of from 2 to 3 in.

*Acknowledgment:*

The Bureau desires to express its appreciation of the courtesy extended by the following companies in loaning equipment for use during the construction of the test pavement: National Equipment Co., Blaw-Knox Co., Heltzel Steel Form and Iron Co., Lakewood Engineering Co., and A. W. French Co.

# DISCUSSION

Mr. Stanton Walker[1] (*subsequently amplified by letter*).—The paper by Jackson and Kellermann constitutes a most valuable addition to the literature. The investigation described is unique in that it has, to a greater extent than any other, combined laboratory accuracy with field facilities to study actual conditions. The results of the tests throw light on many questions which have been engaging the attention of highway engineers, and they deserve the most careful consideration of everyone interested in the use of concrete. The twelve conclusions outlined by the authors touch on problems of current importance and are based on comprehensive and convincing data. All of them are worthy of discussion and the viewpoints of others on them would undoubtedly add to the value of the paper. The present discussion, however, is restricted, for the most part, to a consideration of the authors' conclusion No. 4, which reads as follows:

> The workability of the concrete was greatly reduced by decreasing the maximum size and at the same time increasing the amount of fines in the gravel coarse aggregate, even though the corresponding variations in the percentage of voids in the gravel and the corresponding variations in the ratio of $b$ to $b_0$ for the concrete were small.

That conclusion is of vital interest to the aggregate producer and it is important that its full significance be appreciated. It is based on observations of the appearance of the concrete and on the amount of honeycombing found in concrete slabs, using two sizes of gravel—¼ to 2½ in. (grading A) and ¼ to 1¼ in. (grading B). The percentage of honeycombing provides the only tangible record of the differences in workability. The concrete made with the ¼ to 2½-in. gravel (grading A) showed considerably less honeycombing than that made with the ¼ to 1¼-in. gravel (grading B). It should be made clear that the conclusion is based on the use of the same series of arbitrary proportions for the two sizes of material and that it contains no inference that mixtures cannot be designed for the smaller sizes of gravel which will give entirely satisfactory results so far as workability is concerned. The authors' discussion leading to the conclusion does point out, however, that the proportions used, when measured by the more generally advanced criteria, were such as to lead to the expectation of comparable workabilities. That these expectations were not realized is a clear manifestation of the inadequacy of our knowledge of methods of proportioning materials for concrete.

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[1] Director, Engineering and Research Division, National Sand and Gravel Assn., Washington, D. C.

It is granted that a consideration of the obvious features of the several methods of proportioning which have been advanced and most generally used do not provide an explanation for the apparently wide differences in workability which were obtained, as evidenced by the honeycombing, nor do they provide a tool for determining what proportions would have given comparable workabilities. The straightforward application of the fineness modulus method of proportioning leads to the expectation of slightly greater workability for the mixtures using the coarser gravel, but it offers no explanation for the wide differences which were found. A consideration of the ratio of volume of mortar to volume of voids in the coarse aggregate and of the ratio of the solid volume of coarse aggregate in a unit volume of concrete to that in a unit volume of coarse aggregate ($b$ to $b_0$) shows approximately similar characteristics in these respects for the two sizes of gravel. Nevertheless, a detailed study of the characteristics of the proportions used offers considerable evidence that the mixtures with the $\frac{1}{4}$ to $1\frac{1}{4}$-in. gravel (grading B) were considerably "under-sanded" or "under-mortared" as compared with those using the $\frac{1}{4}$ to $2\frac{1}{2}$-in. gravel (grading A), and that the cause of the greater honeycombing

TABLE I.—SURFACE AREAS OF DIFFERENT SIZES OF GRAVEL AND CRUSHED STONE.

Gravel from Massaponax Sand and Gravel Co., Bowie, Md.; a rounded essentially quartz material; specific gravity 2.61.

Crushed stone from New Martinsburg, W. Va.; specific gravity 2.70.

| Size of Particle (Square Sieves) | Mean[a] Diameter, in. | Gravel | | | Crushed Stone | | |
|---|---|---|---|---|---|---|---|
| | | Number of Particles in 100 lb. | Average Diameter,[b] in. | Surface Area,[b] sq. ft. per 100 lb. | Number of Particles in 100 lb. | Average Edge,[c] in. | Surface Area,[c] sq. ft. per 100 lb. |
| No. 4 to $\frac{3}{8}$ in. | 0.263 | 92 500 | 0.28 | 157 | 108 800 | 0.211 | 201 |
| $\frac{3}{8}$ to $\frac{1}{2}$ in. | 0.441 | 19 400 | 0.47 | 94 | 24 830 | 0.345 | 124 |
| $\frac{3}{8}$ to $\frac{3}{4}$ in. | 0.524 | .... | .... | 79 | .... | .... | 104 |
| $\frac{1}{2}$ to $\frac{3}{4}$ in. | 0.624 | 6 810 | 0.67 | 66 | 8 630 | 0.491 | 87 |
| $\frac{3}{4}$ to 1 in. | 0.882 | 2 152 | 0.98 | 45 | 2 196 | 0.775 | 55 |
| 1 to $1\frac{1}{2}$ in. | 1.255 | 709 | 1.40 | 30 | 792 | 1.09 | 39 |
| $1\frac{1}{2}$ to 2 in. | 1.73 | 319 | 1.85 | 24 | 358 | 1.418 | 30 |
| 2 to 3 in. | 2.40 | .... | .... | 17 | .... | .... | 22 |

[a] Anti-log of average of logarithms of openings of the two sieves.
[b] Assumed as spheres.
[c] Assumed as cubes.

found with the finer gravel may be explained logically on a number of different bases. It is the purpose of this discussion to give a brief description of such studies. They have not been carried far enough to justify offering them as a basis of design; they are given with the hope that further studies along these lines may lead to useful refinements in proportioning methods.

*Surface Areas of Coarse Aggregate:*

Surface area of aggregate has been used as a measure of grading. The principal proponents of this function as a measure of significance in the proportioning of concrete and mortar have been L. N. Edwards[1] and R. B. Young.[2] Comparisons of the surface areas of the $\frac{1}{4}$ to $2\frac{1}{2}$-in. gravel (grading A) and the $\frac{1}{4}$ to $1\frac{1}{4}$-in. gravel (grading B) show a wide difference and seem to offer one reason for the differences in workability. The percentage of honeycomb found for the concrete made with the two gradings of gravel and the two gradings of crushed stone show a good relationship to a simple function of the surface area.

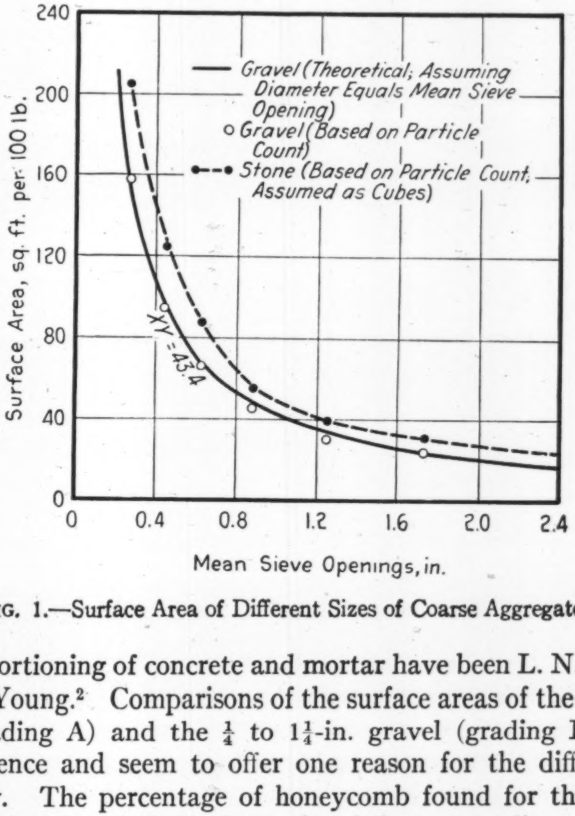

FIG. 1.—Surface Area of Different Sizes of Coarse Aggregate.

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[1] L. N. Edwards, "Proportioning Materials of Mortars and Concretes by Surface Areas of Aggregates," *Proceedings*, Am. Soc. Testing Mats., Vol. XVIII, Part II, p. 235 (1918).

[2] R. B. Young, "Some Theoretical Studies on Proportioning Concrete by the Method of Surface Area of Aggregates," *Proceedings*, Am. Soc. Testing Mats., Vol. XIX, Part II, p. 444 (1919).

R. B. Young, "Mixing Concrete by Surface Areas on Actual Work," *Engineering News-Record*, January 1, 1920, p. 33; also *Canadian Engineer*, January 1, 1920, p. 101.

The surface areas of a gravel similar to that used in the investigation and of a crushed stone from the same source as that used were determined by counting the number of particles of different sizes for a given weight; the surface areas were computed from this information after making an assumption as to the shape of particle. The gravel particles were assumed to be spheres and the crushed stone particles to be cubes. The first assumption is the customary one to make; the second assumption is different from that made by Edwards but, from a detailed consideration of different shapes too complicated to outline here, it seemed to be the most satisfactory, or, at least sufficiently satisfactory, to illustrate the principles under discussion.

TABLE II.—MISCELLANEOUS DATA OF COARSE AGGREGATE.[a]

| | AMOUNTS COARSER THAN EACH SIZE, PER CENT BY WEIGHT | | | |
|---|---|---|---|---|
| | GRAVEL | | CRUSHED STONE | |
| | GRADING A | GRADING B | GRADING A | GRADING B |
| No. 4 square sieve | 100 | 100 | 100 | 100 |
| ⅜-in. square sieve | 85[b] | 60[b] | 90 | 96 |
| ¾-in. square sieve | 67 | 25 | 67 | 82 |
| 1-in. square sieve | 52 | 4 | 52 | 65 |
| 1½-in. square sieve | 27 | 0 | 27 | 35 |
| 2-in. square sieve | 3 | .. | 3 | 4 |
| Fineness modulus | 7.79 | 6.85 | 7.84 | 8.13 |
| Surface area, sq. ft. per 100 lb. | 58.4 | 101.1 | 70.1 | 53.9 |
| Unit weight, lb. per cu. ft. | 108 | 105 | 102 | 101 |
| Voids, per cent | 35 | 36 | 40 | 41 |

[a] Sieve analyses in terms of square openings interpolated from data given in paper in terms of circular openings. Square opening of unity assumed equivalent to circular opening of 1.207 (factor proposed by J. A. Kitts).

[b] Based on supplementary information furnished by W. F. Kellermann.

The accompanying Table I gives the surface areas of the different sizes of particles for the gravel and crushed stone, and outlines some of the steps in arriving at them. The surface areas of the different sizes are shown graphically in the accompanying Fig. 1. Attention is called to the close agreement between the experimental points for the gravel and the theoretical curve based on spheres having diameters equal to the mean[1] sieve opening. It is interesting to note that, while the surface areas of the different sizes of gravel differ considerably from those reported by Edwards, the curve representing them agrees very closely with the one which Edwards reports for different sizes of sand.

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[1] The mean sieve opening was taken as the anti-log of the average of the logarithms of the opening of the two sieves passing and retaining the size under consideration.

Table II gives the sieve analyses and certain other characteristics, including surface areas, for the two gradings of gravel and of crushed stone. Here the sieve analyses have been expressed in terms of square sieves instead of round screens as reported in the paper.

TABLE III.—RELATION OF HONEYCOMBING TO SURFACE AREA OF COARSE AGGREGATE.

Percentages of honeycomb taken from Fig. 15 of Jackson and Kellermann's paper on "Studies of Paving Concrete." Surface areas and volumes of mortar are for 1-sack batches.

| Mix by Volume | Surface Area of Coarse Aggregate, sq. ft. | Volume of Mortar, cu. ft. | Average Thickness of Mortar Film, in. | Average Percentage of Honeycomb |
|---|---|---|---|---|
| GRAVEL GRADING A | | | | |
| 1:2:3½ | 220 | 2.59 | 0.141 | 0.8 |
| 1:2:4 | 252 | 2.64 | 0.126 | 1.3 |
| 1:2:4½ | 283 | 2.65 | 0.112 | 1.9 |
| 1:2:4¾ | 300 | 2.69 | 0.108 | 2.2 |
| 1:2:5 | 315 | 2.71 | 0.103 | 2.5 |
| 1:2:5¼ | 331 | 2.70 | 0.098 | 2.8 |
| GRAVEL GRADING B | | | | |
| 1:2:3½ | 373 | 2.62 | 0.084 | 4.1 |
| 1:2:4 | 426 | 2.65 | 0.075 | 5.6 |
| 1:2:4½ | 479 | 2.72 | 0.068 | 7.3 |
| 1:2:4¾ | 506 | 2.74 | 0.065 | 8.0 |
| 1 2:5 | 533 | 2.78 | 0.063 | 8.6 |
| CRUSHED STONE GRADING A | | | | |
| 1:2:3½ | 251 | 2.60 | 0.124 | 1.0 |
| 1:2:4 | 287 | 2.64 | 0.110 | 1.6 |
| 1:2:4¼ | 305 | 2.65 | 0.104 | 2.0 |
| 1:2:4½ | 322 | 2.67 | 0.099 | 2.2 |
| 1:2:4¾ | 340 | 2.70 | 0.095 | 2.6 |
| CRUSHED STONE GRADING B | | | | |
| 1:2:3½ | 191 | 2.60 | 0.163 | 1.1 |
| 1:2:4 | 218 | 2.64 | 0.146 | 1.6 |
| 1:2:4¼ | 232 | 2.65 | 0.137 | 1.9 |
| 1:2:4½ | 245 | 2.68 | 0.131 | 2.1 |
| 1:2:4¾ | 259 | 2.70 | 0.125 | 2.2 |

This conversion was made by graphic interpolation and by assuming that a square sieve with an opening of unity is equivalent to a circular screen with an opening of 1.207. (This factor has been proposed by J. A. Kitts[1] and several investigators have found it to furnish a quite satisfactory relationship.)

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[1] J. A. Kitts, "Screen Sizes for Concrete Aggregates," *Rock Products*, June 8, 1929, p. 78.

Table III gives the percentage of honeycomb and the surface areas of the coarse aggregate and the volume of mortar for one-sack batches of the different proportions of the two gradings of gravel and of crushed stone. The volume of mortar was calculated on the basis of the average water-cement ratio for all conditions. The average thickness of the mortar film was computed from the surface areas and the volume of mortar. The percentage of honeycombing was taken from the curves in Fig. 15 of Jackson and Kellermann's paper.

The relationship between the percentage of honeycombing and the thickness of the mortar film is shown in Fig. 2. It will be noted that the values for the two gradings of gravel fall on a single curve in a very satisfactory manner. The data for stone of grading A are only slightly off the curve, while those for stone of grading B depart from it a considerable amount. It is believed that these discrepancies can be accounted for very largely by the incorrectness of the assumption for shape of stone particles; different assumptions should have been made for the different sizes, but to have done so would have involved a lengthy discussion outside the scope of this article. The assumptions made serve to illustrate the principles involved. It will be noted that only the surface areas of the coarse aggregates have been considered. It was felt that this was logical since the particles of the fine aggregate had little, if any, influence on the formation of the areas measured as honeycomb. It is recognized, of course, that

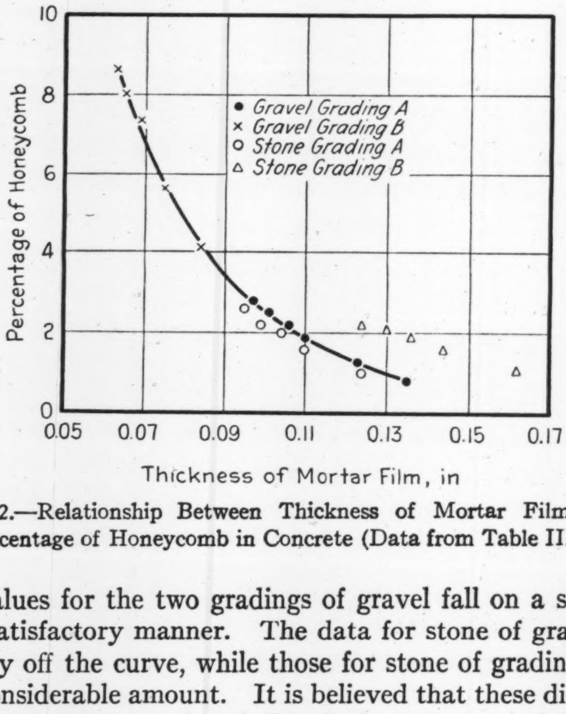

FIG. 2.—Relationship Between Thickness of Mortar Film and Percentage of Honeycomb in Concrete (Data from Table III).

the surface area of the fine aggregate was very large as compared with that of the coarse aggregate.

This brief study of surface areas suggests a problem which seems to deserve further investigation. Further study may show its general applicability as a tool for design.

*Fineness Modulus Characteristics of Aggregates:*

Consideration of the fineness modulus method of proportioning in somewhat greater detail than is ordinarily applied also throws light on the reason for the differences in workability and indicates that these differences were to have been expected for the proportions used. An important and fundamental rule in the application of the fineness modulus to selecting the proportions of different sizes of aggregate, which is not generally known and is rarely applied, is the following:

> The fineness modulus of that portion of the total aggregate finer than any size shall not be greater than the maximum permissible for that size and for the mix in which the material is used.

This rule, which is well known to those intimately identified with the development of the fineness modulus method of proportioning, and perhaps only to those, has not been sufficiently emphasized. The writer does not remember any published discussion of it, although it is inferred in the following rule which appears in the notes to Table 3 of *Bulletin 1* of the Structural Materials Research Laboratory on "Design of Concrete Mixtures," by Duff A. Abrams:

> Sand or screenings used for fine aggregate in concrete must not have a higher fineness modulus than that permitted for mortars of the same mix.

The logic of these rules, and particularly of the more general one, should be apparent. If a mixture of the water, cement and the portion of the total aggregate finer than any size produces a harsh non-workable mix, it is evident that this condition will not be corrected by the addition of coarser material. If the mortar mixture is too harsh it will not become less harsh by the addition of coarse aggregate.

Table IV compares the fineness modulus of the mixtures made with gravels grading A and grading B for the total aggregate and for one intermediate step in the grading. As a basis for comparison, the maximum permissible values of the fineness modulus given in Table 3 of *Bulletin 1* of the Structural Materials Research Laboratory are used. The differences between the fineness moduli for the mixes under consideration and the maximum permissible are shown. A negative difference indicates that the fineness modulus used was lower than the maximum permitted as shown by that table; a positive difference indicates that they were greater.

It must be recognized that these maximum permissible values of fineness moduli for the total aggregate are experimental values based on laboratory compression tests of 6 by 12-in. concrete cylinders. They cannot be applied directly to the investigation under consideration, but it seems entirely reasonable to suppose that they bear a direct relationship to values which would be applicable. A study of the departure of the fineness modulus used from the limiting values outlined in *Bulletin 1* of the Structural Materials Research Laboratory shows some interesting relationships.

TABLE IV.—RELATION OF HONEYCOMB TO FINENESS MODULUS OF AGGREGATE.

| Mix by Volume | Total Aggregate | | | Total Aggregate Finer than $\frac{3}{8}$ in. | | | Percentage of Honeycombing |
|---|---|---|---|---|---|---|---|
| | Fineness Modulus | Maximum Permissible Fineness Modulus[a] | Difference | Fineness Modulus | Maximum Permissible Fineness Modulus[a] | Difference | |
| GRAVEL GRADING A | | | | | | | |
| $1:2:3\frac{1}{2}$ | 5.95 | 6.3 | −0.35 | 3.4 | 4.3 | −0.9 | 0.8 |
| 1:2:4 | 6.1 | 6.2 | −0.10 | 3.45 | 4.2 | −0.75 | 1.3 |
| $1:2:4\frac{1}{2}$ | 6.25 | 6.1 | +0.15 | 3.5 | 4.1 | −0.60 | 1.9 |
| $1:2:4\frac{3}{4}$ | 6.3 | 6.1 | +0.20 | 3.55 | 4.1 | −0.55 | 2.2 |
| 1:2:5 | 6.35 | 6.05 | +0.30 | 3.6 | 4.05 | −0.45 | 2.5 |
| $1:2:5\frac{1}{4}$ | 6.4 | 6.05 | +0.35 | 3.65 | 4.05 | −0.40 | 2.8 |
| GRAVEL GRADING B | | | | | | | |
| $1:2:3\frac{1}{2}$ | 5.35 | 5.5 | −0.15 | 4.1 | 4.3 | −0.2 | 4.1 |
| 1:2:4 | 5.5 | 5.4 | +0.10 | 4.15 | 4.2 | −0.05 | 5.6 |
| $1:2:4\frac{1}{2}$ | 5.6 | 5.3 | +0.30 | 4.25 | 4.1 | +0.15 | 7.3 |
| $1:2:4\frac{3}{4}$ | 5.65 | 5.3 | +0.35 | 4.3 | 4.1 | +0.20 | 8.0 |
| 1:2:5 | 5.7 | 5.25 | +0.45 | 4.35 | 4.05 | +0.30 | 8.6 |

[a] From Table III of paper by D. A. Abrams, "Design of Concrete Mixtures," *Bulletin 1*, Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill.

As pointed out by the authors, the fineness moduli of the total aggregates differed from the maximum permissible in about the same degree for gravels grading A and B. Gravel grading B mixtures are shown to be somewhat harsher on this basis than gravel grading A, but there is not enough difference to account for the considerably greater honeycombing. For example, the 1:2:5 mix, using grading A, has a fineness modulus 0.30 higher than the maximum permitted, while the same mix for grading B is 0.45 higher. Plotting these differentials against the percentage of honeycombing does not show one to be a function of the other. In other words, a single curve cannot be drawn through the points for both gradings of gravel.

Comparisons based on the fineness moduli at intermediate steps in the grading show, however, quite a different story. For the purpose of illustration, the fineness moduli for that portion of the aggregates finer than $\frac{3}{8}$ in. have been studied in Table IV and Fig. 3. For gravel grading A the values of fineness modulus for that portion of the aggregate finer than $\frac{3}{8}$ in. are shown to be well below the maximum allowable. For gravel grading B, all except two of the values are considerably above the maximum allowable and these two are only slightly below. In other words, that portion of the aggregate finer than the $\frac{3}{8}$-in. sieve was obviously considerably coarser in the concrete made with gravel grading B than it was for that made with gravel grading A. Figure 3 shows the relationship between the percentage of honeycombing and the differentials from the maximum permissible fineness modulus of $\frac{3}{8}$-in. material used in these mixes. One curve represents the relationship for both aggregates and indicates that a detailed use of the fineness modulus method of proportioning provides a tool for designing proportions to give comparable workabilities.

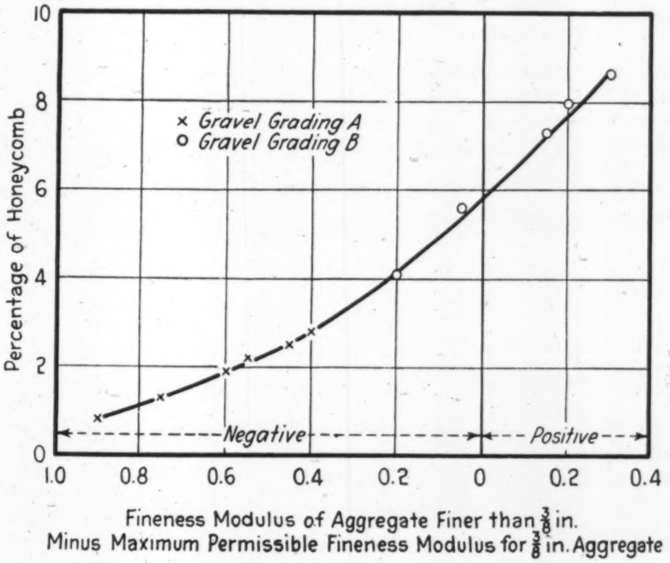

FIG. 3.—Relationship Between Fineness Modulus of Portion of Aggregate and the Percentage of Honeycomb in Concrete (Data from Table IV).

*Effect of Type of Aggregate on Concrete Strength:*

This investigation had for its principal purpose a comparison of the workability of different proportions of fine and coarse aggregate and different gradings of coarse aggregate. It has involved a detailed study of one gravel and one crushed stone. The difference in modulus of rupture obtained for these two materials is marked, although it has not been emphasized in the paper. Lest there be some misinterpretation of the significance of the tests so far as their comparisons of the concrete-making properties of gravel and crushed stone in general are concerned, it seems to be important to point out that the gravel and crushed stone used in the investigation represent materials of widely different physical characteristics. There has been no inference by the authors in the paper that the relationship found between these two materials represents a general one, but it should be of interest to point out that different comparisons, in fact a reversal of conditions, might readily have been obtained by the use of different materials.

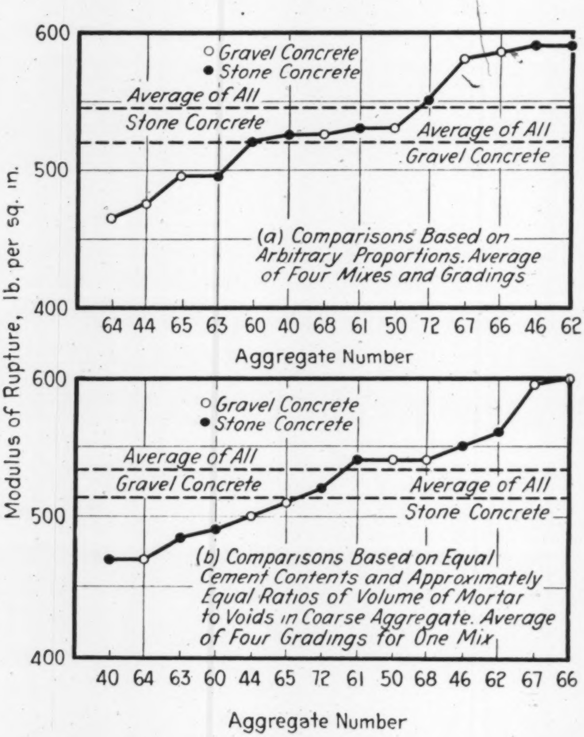

FIG. 4.—Flexural Strength of Concrete Made with Coarse Aggregates of Different Characteristics (Same as Figure A in "Effect of Type of Coarse Aggregate on the Strength of Concrete," *National Sand and Gravel Bulletin*, December, 1929).

Figure 4 summarizes the results of tests reported by W. F. Kellermann in the June, 1929, issue of *Public Roads* on "Effect of Type and Gradation of Coarse Aggregate on the Strength of Concrete." The arrangement of the data are my own and the diagram is taken from my discussion of Mr. Kellermann's paper published in the December, 1929, issue of the *National Sand and Gravel Bulletin.*[1] Gravel No. 44 and stone No. 46 are fairly representative of the types of materials used in the present investigation. It will be observed that they represent almost the extremes of the high and low strengths for a wide variety of materials.

MR. A. T. GOLDBECK.[2]—Mr. Walker has pointed out that the paper by Messrs. Jackson and Kellermann is not intended to be a comparison of the concrete-making properties of gravel and crushed stone in general and I wish to corroborate this. Everyone who has made tests for modulus of rupture has found that the coarse aggregate surely does greatly influence the results. One can obtain a low value for modulus of rupture with some crushed stones, and a high value with others. The same statement may be made with regard to gravel and also slag. I know that Messrs. Jackson and Kellermann did not wish to convey the impression that the test results reported are typical of stones and gravels in general.

I wish everyone would take to heart the definition of the term "workability" given in the paper. I notice that the authors have rather brought out the idea that concrete is workable when it may be placed in a satisfactory manner under job conditions, and with not too much difficulty. That is something that should be taken into consideration by those who are trying to devise some sort of workability test.

The test slabs show that greater honeycombing was obtained when small sized gravel was used as the coarse aggregate than was the case with any other material or with large sized gravel. It has been noticed a number of times that when the coarse aggregate contains too much material of a comparatively small size the workability of the concrete, instead of being increased, is greatly reduced. I am wondering if this phenomenon of the greater honeycombing in small sized gravel concrete might not be explained on the ground that the voids in this type of aggregate are of a comparatively small size and this may render it difficult for the mortar to properly penetrate these voids when the concrete is being placed. We have made some tests on this matter of honeycombing and in a general way we find pretty much the same thing that the authors report with regard to the ratio of volume of mortar to volume of voids. One would expect a smaller amount of honeycombing as the ratio of volume of mortar to volume of voids in the coarse aggregate is increased. That was our experience at any rate.

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[1] See W. F. Kellermann, "Effect of Type of Coarse Aggregate on the Strength of Concrete," *National Sand and Gravel Bulletin*, December, 1929.

[2] Director, Bureau of Engineering, National Crushed Stone Assn., Washington, D. C.

I think the authors' recommendation with regard to the slump of highway concrete should be carefully noted. We have been taught to believe that we should have a very dry mix as measured by the slump test, and in many cases road concrete has been specified to have not more than 1-in. slump. A 2-in. slump is here recommended. Evidently we have been losing strength in the road slab because of the presence of honeycomb and the use of the 2-in. slump will help to overcome this difficulty.

MR. F. C. LANG.[1]—There are two points I wish to discuss. First, I think there is some question as to whether or not the honeycombing shown on those test specimens with a large percentage of coarse aggregate in the concrete would be representative of the honeycombing in a road slab as ordinarily constructed. It has been my observation that as concrete is spread by the mixer bucket some stone rolls to the outside resulting in some segregation in the groove between adjacent bucket dumps. The longitudinal joints spaced 27 in. apart might dam up the flow of the material and reduce segregation or honeycombing. The concrete on which I have observed this segregation had a slump of about 1½ in.

The other point is concerning the use of coarse aggregate containing a large amount of a small size. In 1929 and 1930 we had a special mix in which we used up to 40 per cent of coarse aggregate between a ¼ and ½-in. screen by increasing our mortar content. This mix was harsh and not satisfactory. When the voids in the coarse aggregate are of small size and there is considerable between the No. 10 sieve and the ¼-in. screen in the sand it appears that the larger particles of sand disperse the small stone so that the amount of voids in the coarse aggregate is much less than the space between the stones when placed in the concrete. This year with the same water-cement ratio we increased the amount of stone and decreased the volume of the sand an equivalent amount and obtained a more workable mix by reducing the maximum size of the sand to that passing a No. 10 sieve.

MR. H. F. GONNERMAN[2] (*by letter*).—The data reported in this paper are of interest from several standpoints but particularly in pointing out the possibilities for the occurrence of defects in the finished pavement when coarse aggregate is used in amounts beyond the limits usually considered good practice. A unique feature of the tests was the authors' method of judging the workability by the amount of visible honeycomb in the finished slabs and by their uniformity in strength as determined by tests on beams taken directly from the pavement.

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[1] Professor of Highway Engineering, University of Minnesota, and Engineer of Tests and Inspection, Minnesota Department of Highways.

[2] Manager, Research Laboratory, Portland Cement Assn., Chicago, Ill.

In the construction of concrete pavements, most state highway departments have been accustomed to use a 1:2:3, 1:2:3½, or 1:2:4 mix by volume. During the past few years there has been an effort on the part of progressive highway engineers to effect greater uniformity and economy in pavement construction by more accurate control of the quantities of the constituent materials and by increasing the amount of coarse aggregate. A major factor in achieving these ends has been the practice of separating the coarse aggregate into two or three sizes. These efforts toward improvement in uniformity and economy are commendable so long as uniformity is not subordinated to economy. But careful study will be required to avoid using such large amounts of coarse aggregate as will produce concrete that is honeycombed and therefore non-uniform in strength and lacking in durability.

The authors adopted as a measure of the amount of honeycomb the average of the percentage of the total area of the bottom of the slab showing honeycomb, and the percentage of the total area of the cross-section of the slabs at the break in the flexure tests which showed honeycomb. On this basis, the data showed a gradual increase in the amount of honeycomb with increase in percentage of coarse aggregate in the mix. This was true for all the types and gradings of aggregate used in the tests. In the case of the fine gravel aggregate, designated as gravel grading B, the percentage of honeycombing was markedly greater than for any of the other gradings or types of aggregates used. These results point to the necessity for giving more attention to the selection of aggregate gradings and proportions if honeycomb is to be prevented or reduced to a minimum.

The close relationship observed in all cases between the percentage of honeycomb and the consistency of the concrete as measured by the slump is a feature of the tests that deserves emphasis. The authors point out that while the slump test is not a good measure of workability of concrete where the grading of the aggregate is varied, it proved to be a useful measure of changes in the workability where coarse aggregate of a given type and grading was used. Furthermore, with a given type and grading of aggregate, a distinct relationship existed between the slump and the percentage of honeycombing which developed under the methods of finishing now in common use. When the slump was less than about 2 in. there was a marked tendency toward increase in honeycomb, especially in the leaner mixes. This shows the danger of using very dry concrete with present methods of finishing. While the dry mixes would, for the most part, be rated as workable by laboratory standards, they were not in general found to be workable under the methods of placing and finishing pavement slabs now commonly used.

The authors quite properly emphasize the danger of permitting any condition to exist which will tend toward the formation of honeycomb, since their data clearly showed that such conditions tend also to lower the strength of the slab. To aid in preventing honeycomb it was recommended that a medium consistency, slump of 2 to 3 in., be adopted for concrete paving work where methods of finishing now in common use are employed. The soundness of this recommendation is borne out by the fact that the 2 to 3-in. slump not only gave as high or higher slab strength than the drier mixes but also decreased the tendency toward honeycombing, thereby promoting uniformity and maintaining strength. Of interest to highway engineers particularly in this connection is the authors' conclusion that the amount of honeycomb observed in drilled cores was no measure of the extent of the honeycomb in the pavement slabs.

A noteworthy feature of the tests was the increased variation in modulus of rupture of both gravel and limestone slabs with increase in the amount of coarse aggregate in the mix. The average percentage variation in modulus of rupture was from 1½ to 2 times as great for the leanest as for the richest mixes. These results are particularly significant in view of the fact that the control of the proportions and of other factors on this experimental road was probably better than can be obtained on the average paving job. Other factors affecting the mixture, such as, the amount of water, mixing time, placing and finishing, and the curing of the concrete will not be so thoroughly controlled on the average job as they were on this experimental road. In selecting the proportions, therefore, some allowance should be made for the variations likely to occur on the average job during the progress of the work.

This series of tests is a timely contribution to the subject of pavement practice and testing and the report deserves the careful consideration of all highway engineers. The writer feels that the report would have had added value if, in addition to their recommendation as to the minimum consistency to be employed, the authors had made recommendations as to the maximum amounts of coarse aggregate which in their opinion should not be exceeded in paving work. It is hoped that the investigation will be continued and expanded to include other coarse aggregates and gradings and at least two additional sands, one finer and one coarser in grading than that used in these tests. Such additional studies would afford valuable comparisons not obtainable in the data presented.

MESSRS. F. H. JACKSON[1] and W. F. KELLERMANN[2] (*authors' closure by letter*).—The authors would like to comment briefly on Mr. Lang's statement regarding the effect of the longitudinal separators in controlling segregation during placing.

The construction of each test section required two full 27E batches. Our practice was to dump the first batch with the center of the bucket over one of the outside separators, that is, the separators spaced 27 in. from the side form. The batch was then dumped by drawing back the bucket, a process which distributed the concrete fairly well over a little more than one-half of the entire width of the section. The second batch was then deposited in a similar manner on the other side of the section. The top of each batch thus deposited (and before distribution by the puddlers) would average 3 or 4 in. above the separators. It is difficult, under these circumstances to see how the separator could have influenced segregation, especially if segregation is due to stone rolling over the top and to the outside of the batch. The authors are, therefore, of the opinion that the presence of the longitudinal separators did not tend to induce segregation of the character mentioned by Mr. Lang.

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[1] Senior Engineer of Tests, U. S. Bureau of Public Roads, Washington, D. C.

[2] Associate Materials Engineer, U. S. Bureau of Public Roads, Washington, D. C.

# TESTS OF CONCRETE CONVEYED FROM A CENTRAL MIXING PLANT

By Willis A. Slater[1]

## Synopsis

This paper records the results of tests made to determine the effect on the properties of concrete, of transporting it for 2½ hours in a truck whose container consists of a closed rotating drum without mixing blades. Samples were taken at ½-hour intervals; on these were made tests of slump and composition of the concrete. Compressive strength specimens also were made for tests at different ages.

Within the limits detected by the measurement, the concrete retained the water and all its cement throughout the 2½-hour transporting period. There was considerable decrease in slump and evidence of pulverizing of a considerable quantity of aggregate. The stiffening which occurred within the first hour would improve the consistency of the concrete for almost any purpose. Even after 2½ hours in the drum, the consistency would be quite satisfactory for road and pavement work where the requirements are quite rigid. There was no evidence of segregation during the period of transporting.

The strength of the concrete from samples taken at successive intervals after charging the conveyor drum increased progressively according to the length of time during which the sample had been transported in the conveyor. The average 28-day strength of specimens taken after 2½ hours of transporting was about 900 lb. per sq. in. higher than that taken from the mixer.

## Introduction

In recent years, the use of central mixing plants in concrete construction work has grown rapidly. It has been found that the most difficult problem of the central mixing plant is that of transportation of the mixed concrete from the plant to the construction job. Much progress has been made, from transportation in open trucks to the use of closed conveyors which do part or all of the mixing enroute.

This paper describes tests of concrete transported for periods varying from a few minutes to 2½ hours in a conveyor recently placed on the market by the Clinton Motors Corporation. The tests were made at Fritz Engineering Laboratory of Lehigh University, under the direction of the author. The weighing of all materials was in charge of Mr. C. C. Keyser, and the making of the control cylinders in charge of Mr. G. W. Parkinson, both of the Fritz Engineering Laboratory staff. Mr. W. M. Dunagan of Iowa State College was employed by Lehigh University to take charge of the tests made to determine the composition of the freshly mixed concrete.

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## PURPOSE OF TEST

The purpose of the test was to determine, first, the relation between the strength of the concrete and the time of retention in the drum of the conveyor; and, second, whether a drum without blades would maintain the homogeneity of the concrete during transportation in at least as good condition as that in which it was received from the mixer.

## GENERAL FEATURES OF THE TEST

Every reasonable and feasible precaution was taken to control accurately the quantities of all the materials going into the conveyor so as to afford a basis for comparing the composition determined by tests with the proportions actually used. These precautions extended to allowing for the absorbed and surface moisture in the aggregate in weighing the mixing water.

The operations with the concrete conveyor took place on December 29 and 31, 1930, in five separate runs designated hereafter as series A, B, C, D, and E, in the order named. The average and minimum temperatures for the working hours on these days, based upon data from the local station of the U. S. Weather Bureau were: December 29, average temperature 30° F., minimum 26° F. at 8 A. M.; December 31, average temperature 28° F., minimum 22° F. at 8 A. M. Each of the first four series consisted of charging the conveyor with two cubic yards of concrete, driving the conveyor around the streets for about 2½ hours and taking samples at intervals of about ½ hour. Each of these four series occupied about four hours.

Series E differed from the rest in that the conveyor was driven around with its charge of concrete for only 1 hour 15 minutes instead of 2½ hours and in that no samples were taken, either for determining the composition of the concrete or its strength. It was used principally for inspecting the appearance of the concrete in the drum and during its discharge. An additional test was made in which, for comparison, samples of concrete were taken at intervals from a batch which was mixed in one of the Jaeger mixers used throughout the test.

## DESCRIPTION OF CONVEYOR

The conveyor consists of a truck upon which is mounted a cylindrical drum container of a rated capacity of two cubic yards, into which the concrete is placed through an opening in the surface of the cylinder, about midway of its length. This opening is provided with a water-tight door sealed by a rubber gasket, occupying a groove in the lip of the opening. No leakage was observed during the rotation of the drum.

The drum is smooth on the interior and depends entirely upon friction of the concrete on the surface for any mixing action which may take place. The conveyor is not intended primarily as a mixer, but it is expected to provide enough agitation to prevent segregation of the concrete during transportation. During transportation the drum is rotated at about 8 r.p.m. by a mechanism which is independent of the truck drive, so that the drum may be rotated independently of movement of the truck.

The concrete is discharged through a door at the rear end of the drum, and the drum is stopped with the discharge door at the bottom. In discharging, the front end of the drum is lifted by a power hoist which forms a part of the equipment, so that the drum is inclined at an angle of about 30 deg. At the beginning of the charging operation there was a slight leakage of water due to lack of tightness of the discharge door. The leakage appeared to come from the bottom of the discharge opening, but stopped in a short time and no indication of leakage was seen thereafter until the next time of charging. It was estimated that the water lost in this manner from any charge did not exceed about a quart, that is, about ⅓ per cent of the total amount of water in the charge.

## MATERIALS

*Cement.*—In order to ascertain whether the consistency of the concrete at various times after mixing, or its strength, bore any relation to the time of initial set of the cement, seven different cements purchased in the market were tested for time of initial set, and the two (No. 1 and No. 7) which showed the longest and shortest times of initial set were selected for the test. The lengths of time required for initial and final set were:

| | INITIAL SET, MINUTES | FINAL SET, MINUTES |
|---|---|---|
| Cement No. 1 | 178 | 337 |
| Cement No. 7 | 140 | 322 |

Each value is the average of two tests. Cement No. 1 was used in series A and C, and cement No. 7 in series B and D. With cement

No. 1, 1.6 per cent was retained on the No. 100 sieve, when the cement mixed with sand was screened in water.

*Sand.*—The sand used came from Succusunna, N. J. It was well graded and appeared rather coarser than the concrete sand generally used in this part of the country. The specific gravity of the sand was 2.68. Its weight when rodded dry into a 0.1-cu-ft. measure, was 109.1 lb. per cu. ft., and the voids under this condition amounted to 35 per cent. It had a fineness modulus of 3.15. About 3 per cent of it was retained on a No. 4 sieve and about 2.1 per cent passed the No. 100 sieve.

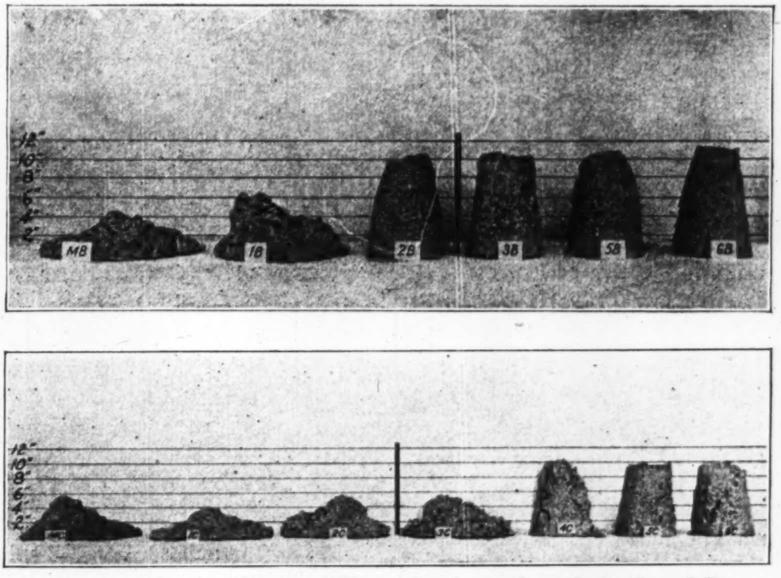

FIG. 1.—Showing Hardened Slump Specimens from Series B and C.

*Gravel.*—The gravel contained about 50 per cent of broken pieces formed by passing the larger pieces through a crusher. All of it passed a $1\frac{1}{4}$-in. sieve. The specific gravity of the gravel was 2.64. Its weight when rodded dry into a 0.1-cu.-ft. measure was 97.9 lb. per cu. ft., the average for three tests. The voids in this condition amounted to 41 per cent.

## CONDUCT OF TESTS

Two small Jaeger mixers were used for mixing the concrete. Eight batches, four in each mixer, were used for each charge of two cubic yards. This kept well within the manufacturer's rated capacity of the mixers. The cement and aggregates were used in the porportions of 1:2.33:4.15 by weight. Seven gallons of water per bag of cement were used, and the absorbed and surface moisture were included as part of the seven gallons. The concrete was mixed $1\frac{1}{2}$ minutes and as it came from the mixer the slump was about 8 in. as is shown at the extreme left in Fig. 1. It was discharged from the mixer into a dump bucket which was hoisted with a crane and dumped into the drum of the conveyor.

Every effort was made to mix the batches rapidly, in order that the length of time between the beginning of the mixing and the completion of the charging of the drum might not be too greatly in excess of that which occurs in commercial operation of the conveyor. However, on account of the necessity of weighing all the materials carefully (48 weighings for each charge of the conveyor), the taking of the samples of concrete, and the handling of the concrete with the crane, it was not possible to reduce the time from the beginning of the mixing until the conveyor was charged to much less than $\frac{1}{2}$ hour. Some improvement was made as the work progressed, as is indicated in that the time required by the successive runs was: series A, 31 minutes, series B, 32 minutes, series C, 23 minutes, and series D, 19 minutes.

Two runs were made on December 29, series A starting in the forenoon with cement No. 1, and series B in the afternoon with cement No. 7. To secure confirmatory data, two more runs were made on December 31, series C in the forenoon with cement No. 1, and series D in the afternoon with cement No. 7.

## METHODS USED IN TESTING

The conveyor was charged with two cubic yards of concrete, for which the quantities of the various constituents were definitely known. Samples were taken of the concrete as it came from the mixer just before it entered the conveyor, and at various time intervals after the conveyor had been charged and placed in operation. One set of samples was used to determine the slump of the fresh concrete and the compressive strength of the concrete after having been cured for various ages. Another set of samples was used to determine the composition of the freshly mixed concrete and thus to discover any tendencies toward segregation.

From each sample used to determine the compressive strength, three slump tests were made promptly after taking the samples, and then eighteen 6 by 12-in. cylinders were made, three to be tested at each of the following ages: 1, 2, 4, 7, 14, and 28 days. In general, one round of specimens (that is, one specimen for test at each of the above-named ages) was made in paper molds, a second round was made in steel molds, and the third round in paper molds. In general, complete sets of 18 cylinders each were taken at about 7 minutes after the conveyor was charged, and at intervals of 30 minutes thereafter, except that the last set, taken about 2½ hours after starting the drum, consisted of only six cylinders. As discussed herein, all the cylinders taken at any one of these times constitutes a "set." There were some exceptions to this routine.

In the conveyor run, series A, 108 cylinders were made. In each of the runs B, C, and D, 114 cylinders were made, making 450 cylinders in all. In taking the samples for all the cylinders of any set, all the concrete for that set was dumped from the mixer or conveyor into a metal scow-shaped container here termed the "boat." The concrete was then remixed with a hoe and shoveled from the boat directly into the cylinder molds. Due to the wholesale nature of the program and the consequent pressure of the work, not much attention could be paid to further remixing of the concrete in the boat before placing it in the molds.

Simultaneously with the sampling for compression specimens, other samples were taken for use in determining the constituents of the concrete. These latter samples (about 5000 g. each) were taken directly from the stream of concrete as it was discharged from the mixer or the conveyor drum. The aim in these analyses was to separate the freshly mixed concrete into its constituents and to determine the amount of each by weighing. The process consists essentially of:

1. Weighing the whole sample in air;
2. Weighing the whole sample in water;
3. Screening the sample in water by the use of the No. 100 and No. 4 sieves nested (allowing the material passing the No. 100 sieve to be washed away);
4. Weighing in water the residue on the No. 4 sieve;
5. Adding to this weight (the residue on the No. 4 sieve) that retained on the No. 100 sieve and weighing in water.

If the specific gravities of the cement and aggregates are not known, their determination constitutes additional operations. All weights are taken in grams. The weights in grams of the coarse and the fine aggregates are found by multiplying by the fraction $\frac{\text{sp. gr.}}{\text{sp. gr.} - 1}$, the weight given in item 4, and similarly that found by subtracting item 4 from item 5. In this fraction, sp. gr. is the specific gravity of the material. The weight of the cement in water is found by subtracting the weight found in item 5 from that found in item 2. Its weight in air is found by multiplying this difference by the fraction $\frac{\text{sp. gr.}}{\text{sp. gr.} - 1.}$ The weight of the water is found by subtracting the sum of the weights of the cement and aggregate as here described from the weight of the total sample given in item 1. If all of the cement and no aggregate passed the No. 100 sieve the determinations would be as accurate as are the weighing operations, but due to the fact that some cement is retained on and some of the fine sand passes the No. 100 sieve, corrections have to be made based upon a predetermination of the percentages so retained and passing.

Several methods have been devised for analysing concrete. That described represents the procedure developed by W. M. Dunagan who was in charge of the determinations of the composition of the concrete. The results of the composition tests help to ascertain whether there was segregation in the conveyor. It is believed that the accuracy of the determinations was sufficient for this purpose.

*Capping and Curing:*

The problem of capping, and of curing, up to the time that the caps were hardened, presented serious difficulties. Owing to the large number of cylinders made on each of the two days (222 and 228 respectively) and the lateness in the day at which the last cylinders were made, it was not possible to cap them on the same day. Also because of the necessity of removing the steel molds for use again, it was not possible to leave them on the specimens while the caps hardened. With the paper molds, it was necessary to cap both ends of the cylinders, and not enough planed steel plates were available to cap them all on one day. It was necessary then to put the specimens in the moist-room awaiting capping.

The specimens tested at 1 and 2 days were stripped at about 24 hours, capped with plaster of Paris or lumnite cement, and tested without storage in the moist-room. The 2-day specimens stood in the open air of the laboratory for 12 to 24 hours before testing. The effect of this early drying probably was compensated for to some extent by the fact that all specimens were immersed and allowed to stand in water for about one hour just prior to testing. All other specimens cast in steel molds were stripped at 24 hours and placed in the moist-room. Those made in paper molds were placed in the moist-room at 24 hours with the molds still on. All specimens (except the 1- and 2-day specimens) were removed from the moist-room at a later date, capped with lumnite cement, and returned to the moist-room later on the same day, except that of series A and B the 4-day specimens remained out of the moist-room a little less than a day and a half, and the 7-day specimens until the time of testing, or a little more than two days. The additional drying was partially compensated for by immersion just prior to testing. Some trouble was experienced in securing satisfactory caps due, apparently, to the short time allowed for the caps to harden before returning the specimens to the moist-room. It was necessary to re-cap about 90 of the specimens with a sulfur compound just before testing.

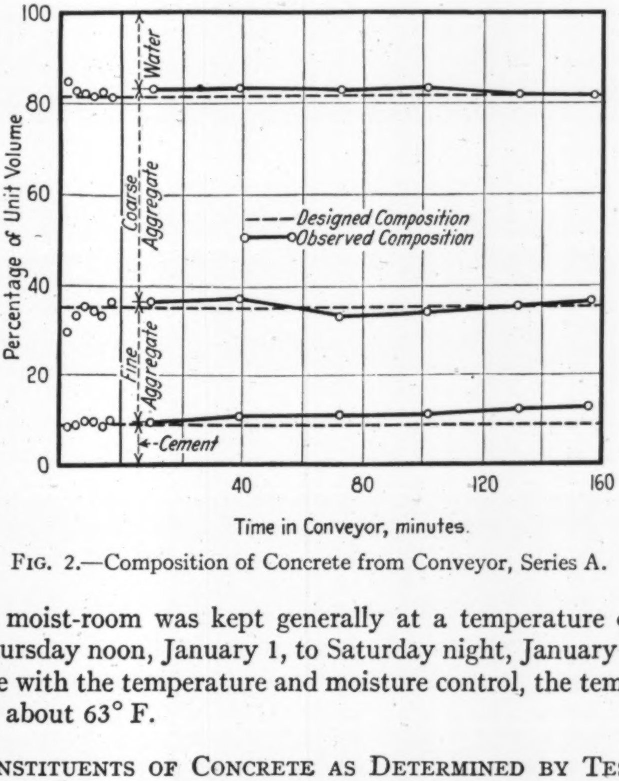

FIG. 2.—Composition of Concrete from Conveyor, Series A.

The moist-room was kept generally at a temperature of 70° F. From Thursday noon, January 1, to Saturday night, January 3, owing to trouble with the temperature and moisture control, the temperature averaged about 63° F.

## CONSTITUENTS OF CONCRETE AS DETERMINED BY TESTS

The determination of the constituents of the concrete was made as described and the results for series B and D and the average for all are plotted in Figs. 2, 3, and 4. The accuracy with which each determination was made may be estimated from the uniformity in results of samples taken simultaneously. In series A, six samples were taken as the concrete came from the mixer. The water content determined from these six samples varied from about 1 per cent higher to 15 per cent lower than the amount of water used in the mix. The apparent cement content in these samples varied from about 12.5 per cent higher to 5 per cent lower, the sand content from 2 per cent higher to 18 per cent lower, and the gravel content from 17 per cent higher to 4 per cent lower than the amount used in the mix.

The average results for all series are plotted in Fig. 4. It is noted that the amount of water determined from the analysis remained practically constant for the concrete kept in the conveyor for different lengths of time. The tests made on the samples taken from the conveyor show a variation in the amount of water from about 0.5 per cent higher to 3 per cent lower than the amount used in the mix.

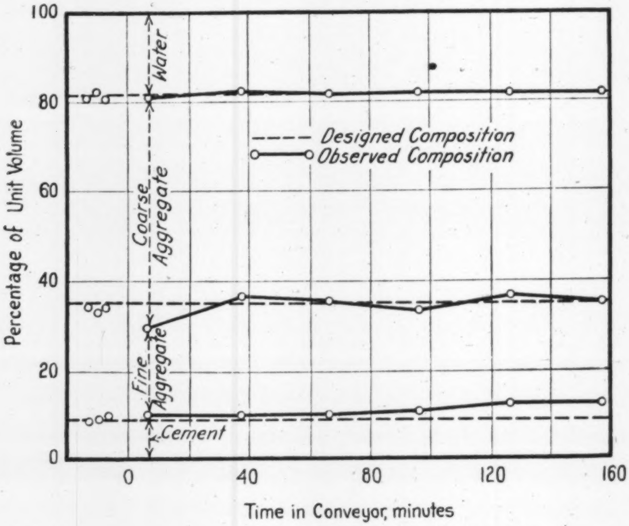

FIG. 3.—Composition of Concrete from Conveyor, Series C.

The observed apparent cement content increased quite regularly during the time in the conveyor. It increased from about 2 per cent higher to 38 per cent higher than the amount used in the mix from the time the concrete was placed in the conveyor until the end of the 2½-hour period. This increase in observed apparent cement content must be due to an increase in the amount of very fine aggregate (passing the No. 100 sieve) during the rotation of the conveyor drum. This is confirmed by the fact that the observed apparent increase in the amount of cement was nearly equal to the decrease in the amount of sand. Evidently the mixing and churning action within the drum was sufficient to pulverize a portion of the ingredients until at the end of the run, the increase in the absolute volume of aggregate passing the No. 100 sieve amounted to about 36 per cent of the absolute volume of the cement. Such an increase in amount of fine particles would be expected to make the concrete appear richer, to decrease the slump, and probably to increase the strength of the concrete.

The average observed sand content varied from about 2 per cent lower to 14 per cent lower than the amount used in the mix. The sand content showed a general downward trend with the increase in length of time in the conveyor, but the results were not so consistent as the increase found in the cement content. The sum of the cement and sand had a slight upward trend, indicating that the quantity of material smaller than the No. 4 sieve increased during the time in the conveyor. The conveyor, therefore, seemed to have some grinding effect upon both the coarse and the fine aggregates. An increase in the amount of aggregate passing the No. 4 sieve would indicate only abrasion of the coarse aggregate, but the increase in the amount passing the No. 100 sieve could indicate pulverizing of both coarse and fine aggregates.

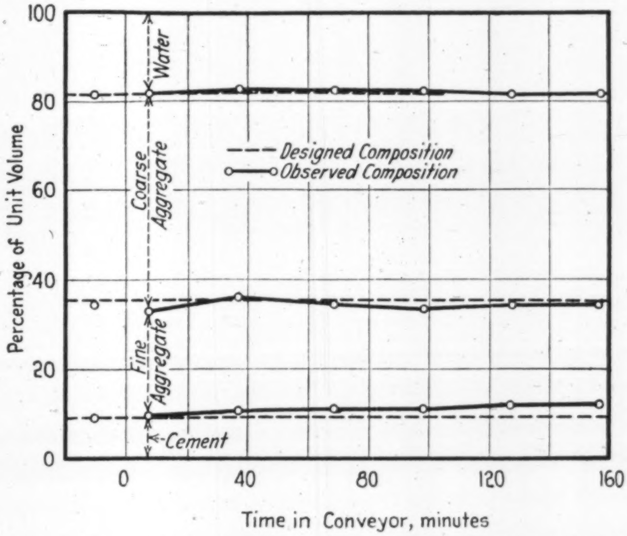

FIG. 4.—Average Composition of Concrete from Conveyor, Series A, B, C, and D.

The average observed gravel content varied from about 4.5 per cent higher to 1 per cent lower than the amount used in the mix. The general tendency was toward a slight decrease in amount of gravel during the time in the conveyor drum. The line representing the sum of the cement, sand, and gravel (see Fig. 4) is practically horizontal for the length of time in the drum. This indicates that the amount of solid materials in the mix remained constant, and adds to the confidence in the accuracy of the results obtained.

To furnish a comparison of the action of the conveyor drum with that of the mixer, a batch of the same proportions as those used in the conveyor was placed in one of the Jaeger mixers and the mixer was run for 1 hour 15 minutes at the same rate as that employed in the mixing for series A, B, C, and D. No slump tests or compression specimens were made. Samples for the determination of the composition were taken at intervals during the operation of the mixer. The results of these tests are given in Fig. 5, from which it will be seen that the sum of the cement and the aggregates remained practically constant. The water also remained constant within the limits of accuracy of the determinations. The apertures for charging and discharging remained open during the operation of the mixer, and there was no protection against evaporation. However, judging from the irregularity of the graph for amount of water, it appears that as much as 10 per cent of the water (about a gallon) could have been lost without being detected by this apparatus.

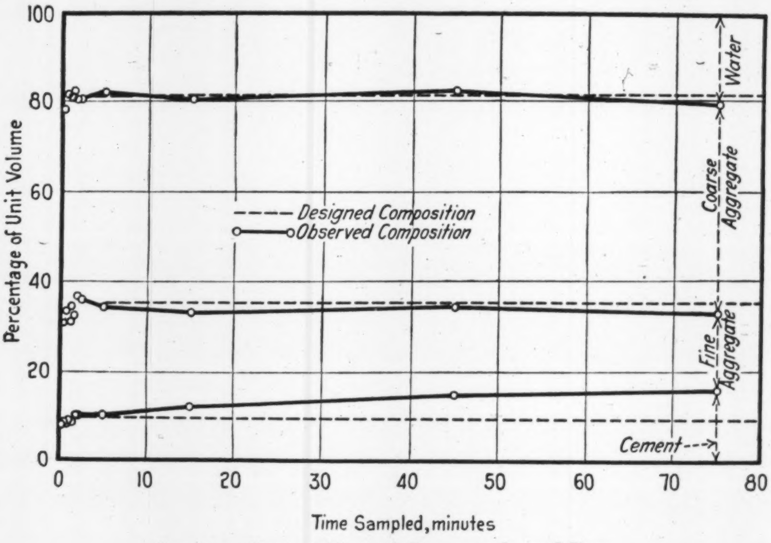

FIG. 5.—Composition of Concrete from Mixer.

The apparent increase in the cement was even more marked than in the conveyor drum as shown in Figs. 2, 3, and 4, indicating a greater pulverizing action in the mixer. Whereas in the conveyor, the absolute volume of the aggregate which was pulverized sufficiently to pass the No. 100 sieve was about 36 per cent, in the mixer the corresponding quantity passing the No. 100 sieve was 72 per cent of the absolute volume of the cement, or twice as much as with the conveyor. Although the time of running the mixer was only about half as long as that of the conveyor drum, the rate of rotation was about twice as great, so that each made approximately the same number of revolutions.

The test was discontinued after 1 hour 15 minutes because the mix had become so stiff that the concrete stuck to the blades and could not be discharged from the mixer. It was supposed at the time that the stiffening was due, at least partly, to evaporation of the mixing water, but the results shown in Fig. 5 indicate that a more important factor was the increase in the amount of aggregate passing the No. 100 sieve.

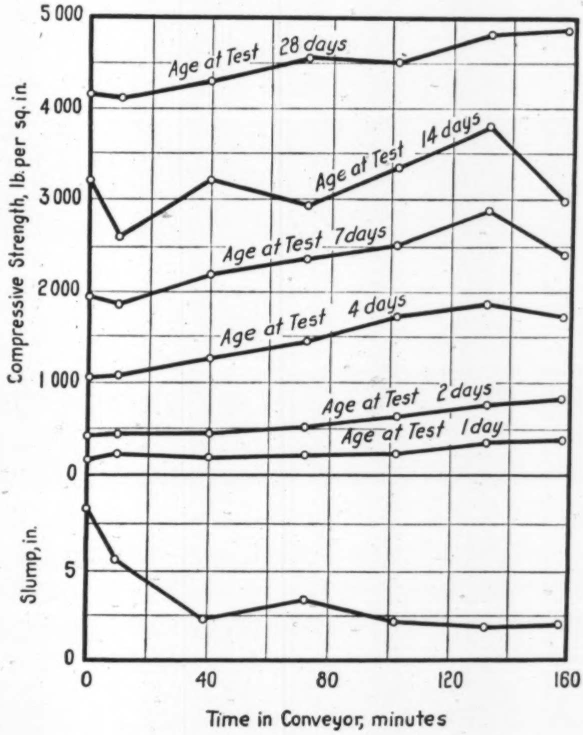

FIG. 6.—Slump and Compressive Strength, Series A.

In all strength curves, the last point for each age represents one specimen only. Other points are averages of three specimens.

The grinding in the conveyor and in the mixer was somewhat unexpected, but the results are consistent with each other. There seems to be no reason for suspicion of the results, but those for the mixer test should be used with caution, since no repetition of the test was made for the purpose of securing confirmatory data.

## CONSISTENCY OF CONCRETE

Three slump tests were made on each of the concrete samples taken for each set of compression specimens, that is, at the time of charging the conveyor and at successive intervals after the concrete had been placed in the conveyor. In Fig. 1 are shown photographs of representative hardened slump specimens taken at successive intervals in series B and C. The slumps thus measured from each sample are shown in Figs. 6 and 7 for series A and C in which each value is the average for three tests made at the time of a given sampling. The average results from all four series are shown in Fig. 8. On the first day of the test, the slump with both cements decreased faster during the first 30 minutes in the conveyor than during the remaining periods. On the second day of the tests, however, both cements showed a more uniform decrease in slump from the time of charging the conveyor until the end of the test. This difference may be due partly to the difference in weather conditions on the two days of the

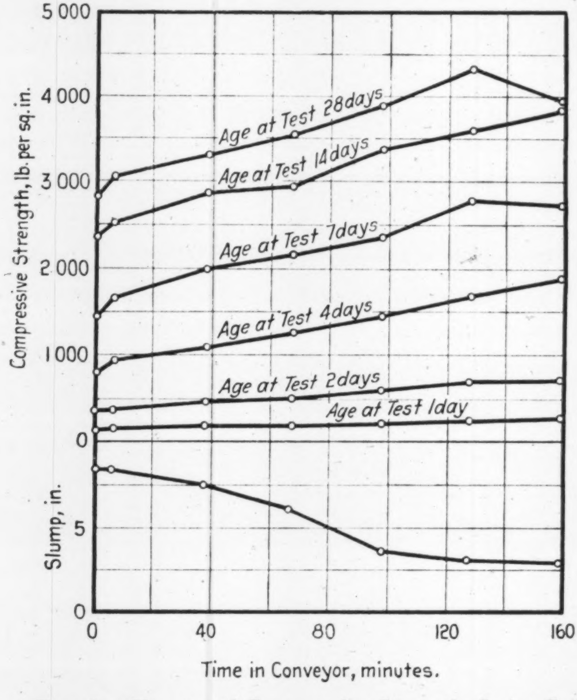

FIG. 7.—Slump and Compressive Strength, Series C.

In all strength curves, the last point for each age represents one specimen only. Other points are averages of three specimens.

tests. On the first day there was a decrease in slump from about 8 to 1$\frac{3}{4}$ in. during the total period in the conveyor, and on the second day a decrease from about 8 to 3 in. during the same period. The average for all series (plotted in Fig. 8) showed a uniform decrease in slump from about 8 to 2$\frac{1}{4}$ in. during the time in the conveyor. This decrease in slump of the concrete during its stay in the conveyor may be due, partly, to the increase in amount of very fine material.

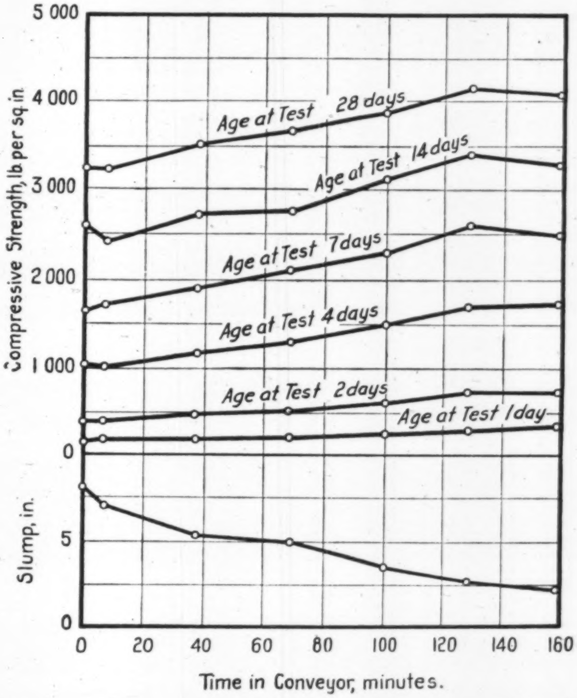

FIG. 8.—Slump and Compressive Strength Averages, Series A, B, C, and D.

In all strength curves, the last point for each age represents one specimen only. Other points are averages of three specimens.

During the run, series C, Mr. Dunagan and the author inspected the condition of the concrete in the conveyor at one of the sampling periods. This inspection was made by opening the "filler door" and observing the appearance of the concrete and its behavior when the discharge door was opened. The coarse aggregate appeared to be suspended uniformly in a fairly stiff mortar with no free water in sight. The consistency appeared uniform and there was no sign of segregation. When the discharge door was opened, the first appreciable movement of the concrete was quite general, but very slight over the greater part of the exposed surface. Soon a somewhat "cone-shaped" portion of the concrete broke away from the rest, due to the flow through the discharge opening. The sample received in the boat resembled closely the surface of the concrete in the drum as viewed through the filler door aperture.

The visual inspection of the concrete at the time of taking each sample indicated that the concrete stiffened and gave the appearance of becoming richer and more plastic during its stay in the conveyor. The workability of the mix probably decreased less than is indicated by the slump test. In fact, as judged by the ease of placing the concrete in the molds, it was in general as workable at the end of an hour as when it came from the mixer and would be less subject to segregation in the forms. Series A showed unusual stiffness at the second sampling, that is, at 37 minutes. The stiffening is indicated in the slump curve of Fig. 6. No other series showed so rapid stiffening, though the observation was that the stiffening was more rapid on the first than on the second day's run. Beyond the one-hour period, up to the end of the run, the concrete remained plastic. It would require more work to place it in reinforced concrete construction than is desireable, but for roads and pavements it seemed entirely adaptable. For the latter purpose it would be vastly superior to the concrete as it came from the mixer. To have secured directly from the mixer, with the same materials, a concrete of as good workability and as high strength as that which came from the conveyor after two hours of transportation, a higher cement content would have been necessary.

The difference between the lengths of time of initial set for the two cements had no effect on the behavior of the conveyor so far as such effect could be observed or measured in these tests.

## COMPRESSIVE STRENGTH

Three 6 by 12-in. cylinders from each sample were tested at each of the ages: 1, 2, 4, 7, 14, and 28 days for each series. The strengths of these cylinders are plotted in Figs. 6 and 7, for series A and C respectively. Each value given is the average for the three companion cylinders except the last, which represents the strength of the one cylinder taken after 2½ hours in the conveyor. The average strengths for all series are given in Fig. 8. It is noted that the strength of the concrete at all ages increased quite consistently with the increase in the time the concrete had been in the conveyor prior to the taking of the samples. The average increase in strength from the time of placing the concrete in the conveyor until the end of the 2½-hour period was nearly 100 per cent for the 1-day tests, 90 per cent for the 2-day tests, and about 80 per cent for the 4-day tests. For the 7, 14, and 28-day tests the increase in strength was nearly the same (700 to 900 lb. per sq. in.) as for the 4-day tests. That is, the time-strength curves were practically parallel for ages above 4 days. The action of the conveyor had, therefore, a beneficial effect inasmuch as the concrete which remained 2½ hours in the conveyor had its strength materially increased. At least a part of the reason for this increase is probably to be found in the fact that the amount of very fine aggregate (amount passing the No. 100 sieve) increased considerably during the time in the conveyor.

## SUMMARY OF RESULTS

The slump was generally about 8 in. as the concrete came from the mixer, but decreased very consistently during the time of retention in the conveyor drum. Even after 2½ hours in the conveyor, the consistency was such that the concrete could have been used quite satisfactorily for road or pavement work, where the requirements are especially rigid.

The amount of water and the sum of the absolute volumes of the cement and aggregates remained practically constant throughout each run. The amount of cement, however, appeared to increase. As this apparent increase was about the same as the decrease in the absolute volume of the sand, it is likely that it was due to the pulverizing of the aggregates, especially the sand.

The apparent increase in cement content in a batch of concrete mixed in a Jaeger mixer for 1 hour 15 minutes was much greater than that in the drum of the conveyor and the stiffening of the batch was correspondingly more marked.

There was no indication of segregation of the concrete in the conveyor drum as observed visually, or as determined by testing the composition of the concrete.

The strength of the concrete from samples taken at successive intervals after charging the conveyor drum increased progressively according to the length of time of transportation.

The difference between the lengths of time of initial set of the two cements used had no effect on the results, so far as such effects could be observed or measured in a test of this sort.

# DISCUSSION

MR. STANTON WALKER[1] *(presented in written form)*.—Mr. Slater has added interesting information on ready-mixed concrete. While his investigation had to do, specifically, with concrete mixed at a central plant and transported in an agitator truck, it is of interest to all ready-mixed concrete producers, whether they operate central plants or truck mixers. The results of his tests should increase still further the reliance placed on the quality of ready-mixed concrete and they should help to dispel what few remaining doubts there may be in the minds of some engineers and architects concerning limitations on the period of time elapsing between the mixing of the concrete and its placement in the work.

The average of the four series of tests reported shows that concrete taken from the conveyor, after being agitated for 160 minutes, had increased in strength over that taken directly from the mixer about 60 per cent when tested at 7 days and about 30 per cent when tested at 28 days. Undoubtedly one of the factors contributing to this increase of strength was better mixing. The beneficial effects of long mixing times is well known, and Abrams' investigation of this factor, described in his paper on "Effect of Time of Mixing on the Strength and Wear of Concrete" presented before the American Concrete Institute in 1918,[2] probably represents the most comprehensive study carried out on the subject. For ten-minute mixing, he found strengths 18 to 122 per cent greater than for one-minute mixing, depending on the proportions and consistency of the concrete. For ordinary proportions, the increases in strength were approximately 25 to 40 per cent.

It has been suggested that the increased strength obtained with longer mixing times is not due entirely to the more thorough mixing but, in part at least, to the period of time during which the water is in contact with the cement and aggregates prior to final placement. There is some evidence to indicate that the increased strengths are caused by the reduction in effective water-cement ratio, resulting from evaporation and absorption of water by the cement and aggregates, and, probably, from acceleration of the hydration of the cement due to the more thorough incorporation of the water. This view is supported by tests carried out on concrete allowed to stand different periods between the time of mixing and molding of the specimens.

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[1] Director of Engineering, National Ready Mixed Concrete Assn., and Director, Engineering and Research Division, National Sand and Gravel Assn., Washington, D. C.

[2] D. A. Abrams, "Effect of Time of Mixing on the Strength and Wear of Concrete," *Proceedings*, Am. Concrete Inst., Vol. XIV, p. 22 (1918).

The "Report on Field Tests of Concrete Used on Construction Work," by W. A. Slater and Stanton Walker,[1] includes a limited amount of data bearing on this question. Concrete allowed to stand one-half hour before molding the specimens gave 28-day strengths about 25 per cent greater than obtained from the specimens molded immediately after discharging the concrete from the mixer. Little further increase in strength was obtained for the one-hour test. Two groups of tests were made which gave essentially the same results. For one group the concrete was kept in an oiled water-tight box and covered with damp burlap, kept damp to prevent evaporation; for the other group the concrete was held, unprotected, in the floor slab form.

Gonnerman and Woodworth in their tests of "Retempered Concrete" reported before the American Concrete Institute in 1929[2] furnish information along these same lines. Tests of concrete allowed to stand for periods up to six hours, protected from evaporation, gave essentially the same strengths. When the concrete was allowed to stand unprotected, so that water was lost through evaporation, an increase in strength resulted up to the time when the concrete ceased to be plastic. These results are not entirely consistent with those quoted above, from which it appeared that the protection of the standing concrete from evaporation had little or no effect on the results.

The results of these tests have a direct bearing on the problem of transporting mixed concrete and furnish evidence that there are two factors at least—increased mixing and longer contact of water with materials—which contribute to an improvement in quality.

Mr. Slater calls attention to the possibility that considerable fine material was added to the mix by the grinding of the aggregates due to the action of the mixer and the conveyor, and that this factor may have contributed to the increased strength. The diagrams included in the paper indicate the likelihood that the fine material was increased and that practically all of the increase may be accounted for by losses from the sand. On my first reading of the paper it was difficult for me to agree that the departure of the "observed composition" curve for cement from the "designed composition" line could be accounted for by the grinding of the aggregates; an abrasive action sufficient to account for the increased fine material seemed unreasonable. Nevertheless, if the samples taken at various times may be considered as definitely representative of the batch as a whole, there seems to be no other logical explanation for the data. It may be said that lack of uniformity of sampling or segregation in the batch would account for these results as well as for the somewhat unusual increases in strength. However, this possibility is made less likely in view of the fact that a study of the data makes it appear that, if these differences were due to lack of uniformity of sampling or to segregation, the assumption must be made that the change in proportions of materials occurred in the mortar, and, specifically, in the mixture of sand and cement. If abrasion takes place to the important amount which seems to be indicated by these tests, it may have very advantageous results from the viewpoint of eliminating undesirable particles from the aggregate. The effect on the sand was considerably more severe than is usually found from running a 500-g. sample of graded sand in a Deval machine for 2000 revolutions with ten $\frac{3}{4}$-in. steel balls added as an abrasive; for this test, the order of magnitude of the loss of material finer than the No. 100 sieve is 2 to 4 per cent, while the conveyor test corresponds to a loss of about 14 per cent, and the mixer test to a loss of about 26$\frac{1}{2}$ per cent.

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[1] W. A. Slater and Stanton Walker, "Report on Field Tests of Concrete Used on Construction Work," *Proceedings*, Am. Soc. Civil Engrs., January, 1925.

[2] H. F. Gonnerman and P. M. Woodworth, "Retempered Concrete," *Proceedings*, Am. Concrete Inst., Vol. XXV, p. 344 (1929).

In order to obtain further data on the question of grinding of aggregates we carried out a few exploratory tests using the Deval abrasion testing machine. Batches of concrete were mixed for one hour in the cylinders of this machine. In order to avoid overlapping of sizes of different materials, the fine aggregate was graded from the No. 14 sieve to the No. 4, and the coarse aggregate from 1 in. to 1$\frac{1}{2}$ in. The fine aggregate was sand consisting essentially of quartz particles. The loss in weight of coarse aggregate due to the grinding action in the Deval machine was small and amounted to less than 1$\frac{1}{4}$ per cent. The loss of weight of the sand as measured by the quantity retained after washing over the No. 28 sieve was about 7 per cent. These tests support Mr. Slater's suggestion that a considerable amount of grinding takes place but they show far less abrasive loss than do his tests. Of course, differences in quality of the sands would affect the results. It seems likely that one hour in a Deval machine, run at 33 r.p.m., should be at least as severe, and probably more severe, than 2$\frac{1}{2}$ hours in an agitator run at 8 r.p.m. It is not clear how the grinding action contributes to the increase in strength, except on account of the greater absorption of mixing water by the finely ground sand and the increase in fineness of the cement.

Investigations of the strength of ready-mixed concrete have been made by several ready-mixed concrete operators. Without exception, they show that hauling periods within practical limits of time and plasticity have no detrimental effect on the concrete, and, in most cases, these experiments show some increase in strength; in general, however, the other tests with which I am familiar do not show as great an increase in strength as found by Mr. Slater, nor as great a decrease in slump for initial periods of about 1 hour. Probably the greater reduction in slump found by Mr. Slater may be accounted for by the unusual amount of grinding which took place.

As early as 1921, the U. S. Bureau of Public Roads carried out a group of tests on "Central-Plant-Mixed-Concrete Tested for Maximum Safe Haul." Concrete was hauled in an ordinary truck body and sampled at various periods up to about three hours. While a decrease in plasticity was found, the strengths showed practically no change.

Some tests carried out by the J. L. Shiely Co., of St. Paul, Minn., and reported by the National Ready Mixed Concrete Association, show comparisons between the strength of concrete sampled at the plant and sampled after being hauled for twelve miles in an agitator truck. The average of three trips showed the strength of the specimens taken after the haul to be about 5 per cent higher than of those taken at the plant. There was a relatively insignificant decrease in slump.

The Boston Concrete Corporation of Boston, Mass., has carried out three comprehensive investigations of the effect of the period of time elapsing between the mixing of the concrete and its delivery to the job. These tests cover different types of agitators, different mixes and different sizes of batch. They have been reported in detail by the National Ready Mixed Concrete Association and a study of them permits of the following general observations:

> "1. Concrete strengths are practically unaffected or slightly increased by periods of agitation up to about 2 hours in either an open-top truck with revolving paddles or a closed-drum type of truck.
>
> "2. The plasticity of the concrete as measured by the slump test decreases with time of agitation. The decrease for a period of agitation up to about one hour is small enough as to be considered negligible under average conditions. For periods of agitation beyond one hour the effect on the slump varies considerably with the conditions of the tests.
>
> "3. An additional observation should be made, on which the results of these tests furnish no evidence, that the decrease in practical workability may not be in proportion to the decrease in slump. The longer mixing may be expected to result in a more homogeneous concrete and one which is relatively easy to place in the forms without segregation."

Several other groups of tests have been made which provide information along these same lines, but it does not seem necessary to summarize them in this discussion. One point should be emphasized with reference to the interpretation of most tests of ready-mixed concrete. There seems to be a tendency to stress the results obtained after 2 and 3 hours. These are important but are not of as much practical significance as the results after ½ hour to 1 hour, which are much more representative of the average haul than the longer periods.

Ready-mixed concrete offers a fertile field for investigation. Better practices will develop with increased knowledge. The National Ready Mixed Concrete Association has been organized by producers of ready-mixed concrete, both plant and truck mixed, to study the problems of the industry and to act as a clearing house for information. The Association stands ready to cooperate with all who are interested in its product, and particularly with specification-making bodies such as this Society. It has already started to develop information which it is believed will be helpful to the producer and user of ready-mixed concrete alike.

A number of researches have been outlined and they will be carried out as rapidly as conditions permit. It has been suggested, for example, that no more mixing time at the plant is required than will serve to produce a uniform mixture; consideration may well be given to the possibility of economizing on mixing time and taking advantage of mixing action of the conveyor. The effect of the time element on the volume changes in concrete may be of considerable importance and represent an advantage for ready-mixed concrete; experience has indicated smaller volume changes in the concrete which has been allowed to stand than in that placed immediately after mixing. If quantitative data on this question are not available, it may represent a profitable line of investigation. Data on uniformity, comparisons between designed strength and actual strength, time of mixing, schedule of mixing operations, methods of proportioning, etc., represent items to which thought is being given and which are essential to taking full advantage of the opportunities for scientific control available to the ready-mixed concrete operator.

THE CHAIRMAN (*Mr. Cloyd M. Chapman*[1]).—I should like to ask if, in these tests, the initial slump was about the same, that is, about 8 in.?

MR. SLATER.—Approximately so.

MR. CHAPMAN.—And if the slump had originally been 3 or 4 in. and any such decrease in slump as was shown in these tests had taken place in the first thirty or forty minutes, the result would have been pretty stiff concrete?

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[1] Consulting Engineer, New York City.

MR. SLATER.—Under the conditions it probably would have had no slump at all.

I want to add one thing about the grinding of coarse aggregate; that is a conclusion that comes only by a process of elimination. We do not know what else it can be. I am always suspicious of any conclusion based entirely on elimination.

MR. M. A. SWAYZE.[1]—I am not at all certain but what Mr. Slater is at least partially right, although he doubts his own conclusions. It is well recognized among cement manufacturers that as cement clinker is ground, more and more particles are made available for hydration within the length of time over which these compression tests were run. The mortar in these tests would closely resemble slurry in a wet process cement plant, while the coarse aggregate would have much the same action as the grinding media in our cement grinding machinery. The coarse particles in cement have little hydraulic action, at least at early ages. In the mixing operation these particles would be ground by the coarse aggregate just as the sand was ground; in fact more readily, since the sand particles are considerably harder than the cement particles. This grinding has made more cement available to hydration, which is the explanation of the increased strength. In other words, while Mr. Slater has not increased the total amount of actual cement he certainly has increased the total *available* cement in that mix by his extended mixing.

MR. W. M. DUNAGAN.[2]—Certain phases of Mr. Slater's paper supplement my report on the Study of the Analysis of Fresh Concrete.[3] I wish to call attention to the fact that there were some seventy-five analyses of concrete made during Mr. Slater's tests; these were made by two men, one of whom was busy carrying samples and the other in doing the analyzing; that these analyses required on an average of about fifteen minutes apiece to complete; that all of the analyses presented in his data are final results, with not one single analysis thrown out because of its being inaccurate. I personally did not perform all of those analyses, but they were performed by means of the methods shown in Appendix V to the Report of Committee C-9 on Concrete and Concrete Aggregates.[4]

I should like also to comment on this matter of increase in the amount of cement. The analyses shown at the left-hand side of Fig. 2 of Mr. Slater's paper are the analyses of samples taken from the mixer; we originally intended to take one out of each of the six batches;

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[1] Assistant Chief Chemist, International Cement Corp., New York City.

[2] Assistant Professor, Civil Engineering Dept., Iowa State College, Ames, Iowa.

[3] W. M. Dunagan, "A Study of the Analysis of Fresh Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 31, Part I, p. 362 (1931).

[4] See Proposed Method of Test for the Field Determination of the Constituents of Fresh Concrete, *Proceedings*, Am. Soc. Testing Mats., Vol. 31, Part I, p. 383 (1931).

however, on the first series of six batches the six analyses checked each other so closely that we decided that only one would be necessary on each run from the mixer. The succeeding analyses from the conveyor were taken as shown on the first six, a correction being made only for the amount of material present in the original aggregate which would pass a No. 100 sieve. Mr. Slater shows analyses to represent the amount of cement present; I should prefer that it be called the amount of material present passing a No. 100 sieve. Since the taking of his data I have averaged the results on a different basis and arrived at a correction curve for computing the grinding that took place over the period of time and find that it is possible by means of that correction curve to adjust those seventy-five analyses and make them all comply with the original batch used.

I have at Iowa State College performed a series of tests such as Mr. Walker outlined. We used a mixture of limestone and one of gravel and found that the same thing occurred, that the rate curves derived in this grinding are very similar for the two aggregates, making it seem that in our analyzing of fresh concrete from central mixing plants, when being agitated for a period of time, that it will be possible to arrive at a correction factor for the length of mixing with some regard to the slump. Such a curve will be quite general in nature but such information as we have indicates that with acceptable material this grinding occurs at quite a definite rate for most materials. At Lehigh University the mixing was carried on over a period of 75 minutes in the open mixer; the reason it was not conducted three hours as in the conveyor was that in 75 minutes the mix became so stiff that it hung in the mixer and had it not been stopped at that time would have had to be chiseled out of the mixer, the slump reducing much more rapidly in the mixer than in the conveyor.

As a final word concerning the method of the test for analysis of fresh concrete I should like to say that it was precisely for such studies as that of Mr. Slater and others on the composition of concrete, during mixing and after its placing, with a view of determining facts to help in evaluating its quality after it has set, that the method was developed.

MR. E. C. SHUMAN.[1]—In respect to the question of the grinding of the aggregate, some time ago I had the opportunity to study some features of concrete mixers. In these studies, some tests were conducted on a 2-yd. mixer operated in the usual manner except that as a matter of economy no cement was put into the batch. The mixer was run about fifteen minutes, the materials (fine and coarse aggregate and water) then discharged and reloaded into the mixer for further tests. At the end of another fifteen minutes the materials were again discharged. By this time the amount of grinding of the aggregates had been so great that the mass was remarkably plastic, much more so than many mixes I have seen with cement in them.

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[1] Research Engineer, Pennsylvania-Dixie Cement Corp., Richard City, Tenn.

MR. D. A. ABRAMS.[1]—If we express the gain in concrete strength as shown in Fig. 8 of the paper in terms of percentages of the initial strength, we secure the following increase for each 1 minute in the conveyor:

| | |
|---|---|
| 1 day | 0.4 per cent |
| 2 days | 0.6 " |
| 4 " | 0.6 " |
| 7 " | 0.4 " |
| 14 " | 0.3 " |
| 28 " | 0.2 " |

A recent investigation of effect of fineness of cement carried out by the speaker showed the following increase in concrete strength for each 1 per cent increase in fineness as measured by the amount passing the No. 200 sieve:

| | |
|---|---|
| 1 day | 2.8 per cent |
| 2 days | 3.3 " |
| 3 " | 3.0 " |
| 7 " | 2.4 " |
| 28 " | 1.7 " |
| 3 months | 1.7 " |
| 1 year | 1.3 " |

These were compression tests of 6 by 12-in. concrete cylinders; mix 1:4, water-cement ratio 0.80. The cements used were ground from the same clinker to finenesses ranging from 50.0 to 95.6 per cent; however, in computing the above values only cements ranging in fineness from 80.0 to 95.6 per cent were considered.

Increased time in the conveyor had an effect on concrete strength similar to that produced by fine grinding of the cement. Since the two groups of tests were made on different materials and mixtures we cannot compare the strength directly; however, it appears that each 6 minutes in the conveyor produced about the same increase in concrete strength as if the cement had been ground finer by 1 per cent through the No. 200 sieve. The increased concrete strength in Mr. Slater's tests is probably due to a number of causes, but there is strong evidence that fine grinding of the cement was an important factor.

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[1] Consulting Engineer, New York City.

# CRYSTALLINE TALC AS AN ADMIXTURE IN CONCRETE

By Frank R. Wicks[1]

## Synopsis

This paper discusses the general theories regarding the use of dry, powdered admixtures in concrete and their effect upon the water-cement ratio.

Particular attention is given powdered crystalline talc as a type of admixture material most suited to the theoretical properties desired in admixtures.

The author points out that crystalline talc has been extensively used with excellent results, particularly in structures where the architectural designs called for exposed concrete having intricate surface detail poured in monolithic construction.

## Introduction

Opinions differ as to the effect of admixtures in concrete, and many of the factors involved are difficult to reduce to exact figures, so any article on the subject is certain to bring forth much discussion. It is with sincere respect for the knowledge and beliefs of other engineers, who have had extensive experience in the handling of concrete, that the author approaches the problem of admixtures in general and crystalline talc admixture in particular.

Each successive year has witnessed advancement in the knowledge of the art of making concrete. Crude methods and personal opinions have been gradually reduced to systematic routine and measurable facts. The manufacture of hydraulic cements seems to be reaching a state of near-perfection; the problems of selection and grading of aggregates have been made more simple; the epochal demonstration of the water-cement ratio law is recognized as a definite step in advancing the knowledge of the fundamentals of concrete construction. Now comes a further step, that of the development of the finer qualities of man-made stone, maximum strengths, maximum densities, greater uniformity and a nearer approach to the inherent permanence and beauty of natural stone.

In this latter improvement in concrete and the more general use of concrete for its artistic and architectural possibilities, the judicious use of suitable admixtures has played an important part. In addition to obtaining the greatest possible strength that could be derived from a given set of ingredients, it has been increasingly desirable to insure the complete filling of intricate forms and to produce a concrete having uniform texture and presentable appearance, such as would require little or no surface treatment after completion. Many engineers have found that good admixtures are of substantial assistance in such work.

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[1] Consulting Engineer, Pacific Coast Talc Co., Los Angeles, Calif.

*Meaning of "Admixture":*

The use, in concrete, of certain materials, other than cement, aggregates and water, particularly finely ground inert substances, is not new, but it is only within the last few years that the term "admixture" has come into general use. The term is not yet definitely defined in its application to concrete construction, but seems to be considered by the American Concrete Institute[1] to "include everything added to concrete (besides the essential ingredients—cement, water and aggregates) *in the mixer*, without restriction as to the object of its addition—".

The generally accepted meaning of an admixture may perhaps be more nearly defined as any substance, other than water, added to concrete in relatively small proportions for improving the workability of the mix or for beneficiating some other essential property of the concrete.

Following out this same general line of reasoning, the 1929 report of Subcommittee XV on Admixtures of the Society's Committee C-9 on Concrete and Concrete Aggregates[2] presents the first logical classification of commercial admixtures. This report covered 81 commercial admixtures, and developed a classification of six major groups or types of such materials.

*When to Use an Admixture:*

From this array of available admixtures, and many others which the committee did not report upon, the selection of the correct material for a specific purpose becomes a major problem. Those charged with the responsibility of specifying ingredients and those engaged in the placement of concrete are sometimes undecided as to the advisability of using an admixture. There is a very universal desire to get into concrete the best materials and workmanship that can be had for the money available, but the determination as to what is best is sometimes a difficult task.

To what extent the ordinary materials, graded rock, sand, cement and water will go toward the production of an ideal concrete should be given first consideration. When it has been shown that these standard ingredients will not produce fully satisfactory concrete, or do not produce as many units of strength per dollar of cost as can be accomplished through the use of an admixture, it is then time to consider the use of admixtures, either for improved workability or improved quality or lowered cost, or to secure some special quality which the standard ingredients do not seem to be able to develop.

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[1] Editorial, *Journal*, Am. Concrete Inst., November 1930, Vol. 2, No. 3, pp. 6 and 7.

[2] *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part I, p. 305 (1929).

In selecting admixtures, as well as other materials for use in concrete, one must consider not only the immediate effect upon workability, consistency and volume of concrete, but the ultimate strength and condition of the concrete after it has attained considerable age.

An admixture, to be useful, must be of such character as to result in some definite improvement in the quality or character of the concrete, or to be of substantial assistance in the placement without adversely affecting the other properties of the permanent structure. No admixture is a general panacea, nor will the use of any admixture, or all of them, wholly cure the defects of improper grading, poor cement, excessive amounts of mixing water, or irregularity in placement. On the other hand, there are a number of commercial admixtures which can be used with safety and which produce beneficial results in concrete construction that would not be so easily or cheaply attained if no admixture were included.

The development of definite principles governing the application of admixtures is yet far from complete. This paper is an attempt to contribute toward such development, and to so analyse the problems involved as to assist in working out correct answers to these questions.

## POWDERED MATERIALS AS FILLERS

Any finely powdered material added to concrete, whether it be of the active or pozzuolanic character or if it be totally inert chemically, must in all probability serve to fill voids or pores in concrete in exact proportion to the absolute volume of the powdered material used. The filling of pore spaces means increased density and this in turn means higher strength and longer life, assuming all other conditions equal. Unfortunately, however, the introduction of most powdered materials necessitates the addition of extra mixing water because of the absorptive character of the admixture, which to a large extent reduces the beneficial results that would otherwise be accomplished.

## WORKABILITY AND WATER ABSORPTION

Many powdered admixtures have a favorable effect upon the workability of the mix and tend to reduce segregation during placement, but the individual characteristics of these powdered substances differ so widely as to produce divergent effects in the permanent structure. There has long been a justifiable objection to the use of silt, loam and many clays, because of the probability of contamination with organic substances and because their highly absorptive nature tends toward the use of greater proportions of water for mixing.

Some nationally used admixtures have such ability to improve the consistency and workability of concrete as to largely offset the detrimental effects of the extra water which must often be used with the admixture. Other materials make a favorable impression upon the man who mixes and places the concrete, but prevent the concrete from attaining as high strength as could be desired because of the necessity of using higher proportions of mixing water or because of adverse chemical action of some kind. It has been so conclusively shown that the ultimate strength of concrete bears a direct relation to the ratio between the cement and the total water in the mix at the time of placement, as to lend serious doubt as to the advisability of using any water-absorbent materials for the production of workability in concrete.

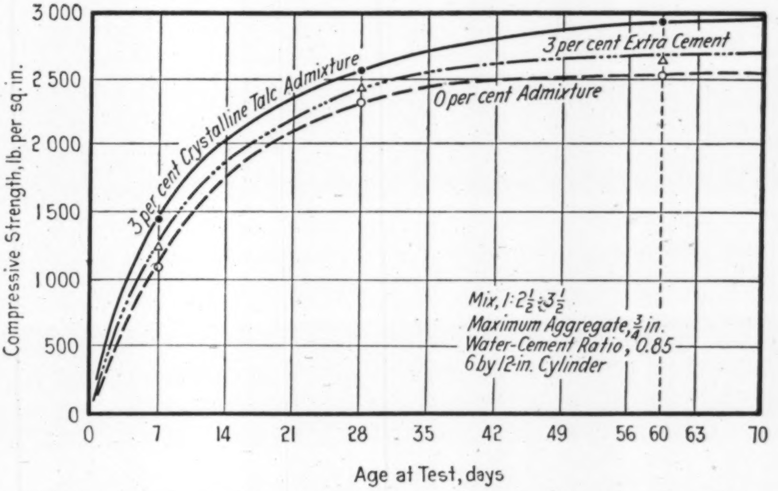

FIG. 1.—Showing Relative Strength of Concrete Using Crystalline Talc and Extra Cement as Admixtures.

**Each result shown is average of five tests.**
**Cylinders made and tested by Osborne Laboratories, Los Angeles.**

To a somewhat lesser extent, the same condition is obtained by the use of small amounts of extra cement as an admixture for improvement of workability, as it simply means additional water-absorbing material, which in turn calls for some extra water. While it is unquestionably true that more cement means a richer mix, and richer mixes are usually more workable and develop stronger concrete, it is the author's experience that an equal weight of a suitable admixture, within the critical range of the particular admixture, will be much more effective in producing workability and uniformity in concrete than can be obtained with portland cement. This conclusion is based upon observation of a large number of construction jobs wherein the concrete mix has been changed from plain concrete to admixture concrete, or from admixture concrete to plain concrete, with and without changes in the proportioning of aggregates in the mix, which had the effect of altering the relative amounts of cement in the concrete. The effect of admixtures is less noticeable, so far as appearances go, when plastic cements are used, and is greater when using coarse ground cement or cement which has been in storage for a considerable time before use.

To determine the relative strengths developed when using extra cement as compared with an equal weight of crystalline talc, a short series of laboratory tests were conducted as shown in Fig. 1. In this test, the same amount of water was used for making each cylinder, which was individually mixed by hand.

## CO-MIXTURES

The pozzuolanic materials, which are either natural cements or portions of natural cements, such as trass, volcanic ash, etc., are usually used more as co-mixtures than as admixtures; in other words, these materials are used in relatively large proportions to replace a part of the portland cement in the mix and thus their presence produces something different from ordinary portland-cement concrete.

It has seemed to the author that when quicklime or hydrated lime is used in concrete in proportions approximating eight to ten per cent of the cement used, the lime should really be considered a co-mixture, as the resulting product is in reality a mixture of portland-cement concrete and sand-lime mortar. Thus it would appear that more definite distinction should be made between the chemically active co-mixtures and those substances which are properly called admixtures.

## CRYSTALLINE TALC AS AN ADMIXTURE

Bearing in mind the fundamental principles of making concrete, it is the opinion of many engineers that the use of finely powdered crystalline talc is worthy of thoughtful consideration. This material, a natural mineral, differs from other admixture materials in a number of respects.

Crystalline talc possesses four natural properties which seem essential to a successful admixture. It is chemically inert; it is water-repellant; it has the same volume wet or dry; and it has the peculiar ability to lubricate because of its natural slip or grease-like smoothness.

When used in proportions of one to three pounds per sack of cement in a concrete mix, crystalline talc acts much the same as graphite in oil, adding body to the lubricating medium; it thus improves the working condition of the mix without the necessity of extra water, or it replaces approximately its own weight of water without loss of workability.

Crystalline talc does not readily become wet or coated with water. Its greasy surface tends to remain essentially dry even when submerged in water. Hand-size specimens may be subjected to dripping water for a considerable time without showing appreciable absorption of water. Powdered talc will float on water almost indefinitely, even though it has a specific gravity of 2.6. Talc on the surface of water will turn the meniscus down just as will the presence of oil or grease. A drop of water may be coated with dry talc powder and rolled around like a globule of mercury.

To determine the relative volume of powdered talc wet and dry, 50 g. of powdered crystalline talc may be packed into a 20-cc. graduate cylinder. If then the same talc be made up into a stiff paste with water and allowed to stand for an hour it may be returned to the graduate and will be found to occupy practically the same volume as when dry.

During the making of concrete, a certain portion of the mixing water is normally absorbed by the aggregate and is thus withdrawn from the mix and made unavailable for lubrication. The more porous or pervious the aggregate, the greater the absorption. When crystalline talc is added to the mix, an extremely thin coating of this talc powder seems to form on the surface of the aggregates, and the talc, being water repellant or water resistant, serves to temporarily retard or delay the passage of water into the aggregate, thus leaving a greater portion of water available for lubrication during the mixing, transportation and placement. After talc has served this purpose, it remains in the mass as a permanent, water-resistant, inert filler, which does not in any way alter the rate of hydration or interfere with the normal development of tensile and compressive strengths.

## CHEMICAL AND PHYSICAL PROPERTIES OF TALCS

Pure talc is hydrous magnesium silicate. The talcs having distinctly crystalline structure belong to the classification known as steatite, and should be distinguished from the massive or soapstone varieties, and must not be confused with the so-called talcy clays.

The principle production of the crystalline talc is from San Bernardino County, Calif. This type of talc is white, both in the lump form and after pulverizing. It has a hardness of 2 to 3 on the standard scale, and shows a very distinct crystal under the microscope and to the unaided eye as well. It is presumed to have originated from the mineral tremolite and retains much of the original crystalline structure of the tremolite as well as a portion of the calcium silicate contained in the parent rock. Talc is one of the least alterable and therefore the most stable or chemically inert of the mineral species.[1]

The average chemical analysis of crystalline talc used for the manufacture of admixture in California is about as follows:

| | | |
|---|---|---|
| Silica | 59.5 | per cent |
| Alumina | 3.7 | " |
| Iron oxide | 0.6 | " |
| Lime | 5.3 | " |
| Magnesia | 26.6 | " |
| Ignition loss | 4.3 | " |
| Total | 100.0 | per cent |

The approximate natural ingredients, making up the above analysis, are about as follows:

| | | |
|---|---|---|
| Talc, hydrous magnesium silicate | 74 | per cent |
| Tremolite, calcium-magnesium silicate | 14 | " |
| Pyrophylite, aluminum silicate | 5 | " |
| Magnesite, magnesium carbonate | 2 | " |
| Limestone, calcium carbonate | 2 | " |
| Iron hydrates | 1 | " |
| Free silica | 2 | " |
| Total | 100 | per cent |

It should be noted that the material contains no free lime or free magnesia, such as would be objectionable in portland-cement concrete. The total acid-soluble ingredients amount to less than five per cent of the total and none of the material is water soluble.

This is of the same class of talc which is extensively used for the manufacture of cosmetics and is used in the ceramic industries as one of the principal materials in certain kinds of tile.

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[1] F. W. Clarke, Data of Geochemistry, U. S. Geological Survey *Bulletin 695*, p. 607 (1920).

It is customary to grind crystalline talc for admixture purposes to approximately 99.0 per cent through a No. 200 sieve. This appears to produce better results than either coarser or finer grinding, but data in this respect are not yet complete.

## TESTS OF CRYSTALLINE TALC AS AN ADMIXTURE IN CONCRETE

It is so difficult to measure workability of concrete in the laboratory in such a way as to be comparable with actual construction conditions that most of the laboratory work has been performed with an equal water-cement ratio for both plain concrete and the concrete containing talc admixture even though it has been quite conclusively shown by construction experience that a lesser proportion of water is usually needed when crystalline talc is used.

TABLE I.—PERMEABILITY TESTS OF CONCRETE DISKS.

8 in. diameter; 3 in. thick.
Age 14 days. Water pressure 70 lb. per sq. in.

| | Series A | | Series B | |
|---|---|---|---|---|
| | Crystalline Talc Admixture | | Crystalline Talc Admixture | |
| | None | 3 per cent | None | 3 per cent |
| Cement, by volume | 1.00 | 1.00 | 1.00 | 1.00 |
| Sand, by volume (110 lb. per cu. ft.) | 1.97 | 1.97 | 3.94 | 3.94 |
| Graded gravel (103 lb. per cu. ft.) | 2.73 | 2.73 | 3.12 | 3.12 |
| Fineness modulus of mix | 5.1 | 5.1 | 4.6 | 4.6 |
| Water-cement ratio | 0.93 | 0.93 | 1.45 | 1.45 |
| Slump, in | 3.0 | 3.5 | 3.0 | 2.5 |
| Average leakage, cc. of water through 6-in. diameter center ring: | | | | |
| Initial period, 5 minutes | 0.37 | 0.40 | 8.20 | 2.71 |
| Total period, 30 minutes | 0.90 | 0.67 | 80.50 | 25.80 |
| Highest total leakage for one disk | 1.50 | 1.20 | 99.80 | 43.50 |
| Minimum leakage shown by one disk | 0.60 | 0.40 | 61.2 | 13.00 |
| Computed average leakage, gallons per 100 sq. ft. per hour | 0.241 | 0.179 | 2.155 | 0.690 |

If crystalline talc is added to normal concrete mixes without making any change in water content or other ingredients, the mix appears more pasty and becomes more workable, but the consistency as measured by the slump cone does not change appreciably. On a recent test of 1:6 concrete on building construction, with as nearly constant conditions as could be maintained, the slump of plain concrete averaged $4\frac{1}{16}$ in. for seven trials, while the slump of concrete containing $2\frac{1}{2}$ lb. of crystalline talc per sack of cement and with the same water content, averaged during seven succeeding trials $3\frac{3}{4}$ in., or a reduction of $\frac{5}{16}$ in. After reducing the proportion of mixing water one-fourth gallon per sack of cement, the slump was further reduced to an average of $3\frac{5}{8}$ in. In this test, the water-cement ratio was not known, as the moisture in the aggregate was not determined.

Flow table tests have been so inconclusive as to be of little assistance in comparing plain concrete with concrete containing crystalline talc. As a rule the talc concrete shows less flow on the flow table than plain concrete in rich mixes but more flow in lean mixes.

When comparing plain concrete with concrete containing crystalline talc and having the same water-cement ratio, the mix containing

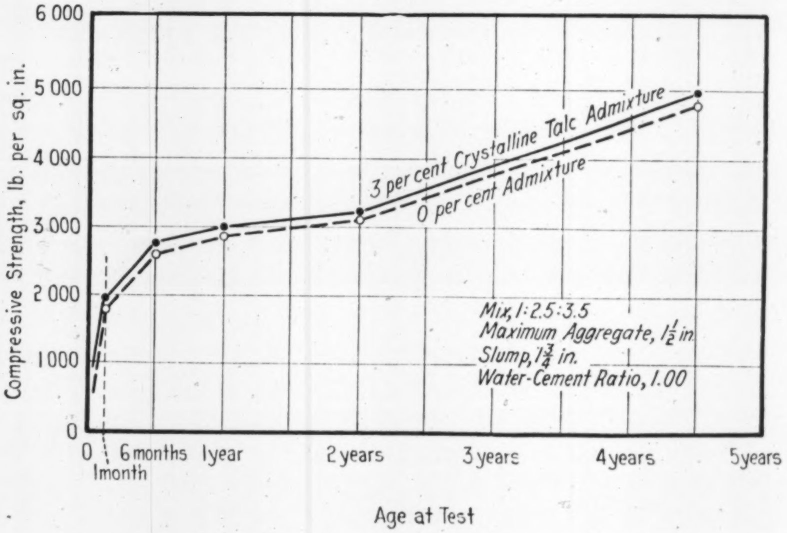

FIG. 2.—Showing Relative Strength of Concrete, With and Without Crystalline Talc Admixture.

Each result shown is average of three tests. 6 by 12-in. cylinders made and tested by Osborne Laboratories, Los Angeles.

the talc will be more easily placed in forms, will be slightly more dense, as indicated by permeability tests shown in Table I, and will be much more uniform, as indicated by observation and photographs of completed structures. The talc concrete will usually develop about five per cent more compressive strength at 28 days, as indicated by numerous tests made during the progress of construction. Somewhat greater difference is shown in the laboratory results as shown by Figs. 1 and 2 and the upper curves of Fig. 6, as well as in the mortar tests shown in Fig. 3.

Figure 2 shows the comparative results of a series of cylinder tests begun in September, 1926, to determine the relative compressive strengths of concrete with and without crystalline talc admixture, all other conditions being the same. Forty 6 by 12-in. cylinders were made for this series, each being individually mixed by hand from weighed ingredients, as follows:

| | |
|---|---|
| Cement, a local brand | 3.65 lb. |
| Sand, local washed, commercial grade | 11.0 lb. |
| (Fineness modulus, 2,73; silt or loam, 1.4 per cent) | |
| Crushed river gravel, graded ¾ in. to 1½ in | 13.0 lb. |
| (Fineness modulus, 7.42) | |
| Water | 2.42 lb. (1098 cc.) |
| (Equals water-cement ratio of 1.00) | |
| Crystalline talc, when used | 0.11 lb. (50 g.) |
| (Equals 3 per cent of weight of cement) | |

The results have shown a higher average strength in concrete containing crystalline talc at every age tested, with a difference of 141 lb. per sq. in. at 4½ years. This difference is not great, but is in line with the general trend of short-time tests.

Attention is called to the fact that in this series the cylinders were kept in moist storage until tested, and that the strength of both plain and talc concretes at the age of four years was more than double the 28-day figures, indicating what may be expected of good concrete if the proper curing conditions can be maintained.

The density of concrete seems to be invariably improved by the use of crystalline talc admixture. Twenty-four concrete disks were made by one of the commercial testing laboratories and these were tested for percolation with the results given in Table I. The testing engineer reporting these results, stated: "The tests are comparative and great care has been taken to have all the conditions identical for the various sets made with talc and without talc. All ingredients were weighed and conditions of mixing, curing and handling the same."

### *Discussion of Previous Reports on Admixtures:*

Other reports on powdered admixtures presented before the Society[1] deal principally with the effect of relatively large proportions of powdered materials, hydrated lime, diatomaceous earths, clays, etc. These do not include tests on crystalline talc.

In these reports, the admixtures used replaced equal quantities of cement instead of being used as an additional material such as is

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[1] D. A. Abrams, "Effect of Hydrated Lime and Other Powdered Admixtures in Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. XX, Part II, p. 149 (1920); and D. A. Abrams, "Tests of Powdered Admixtures in Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part II, p. 618 (1929).

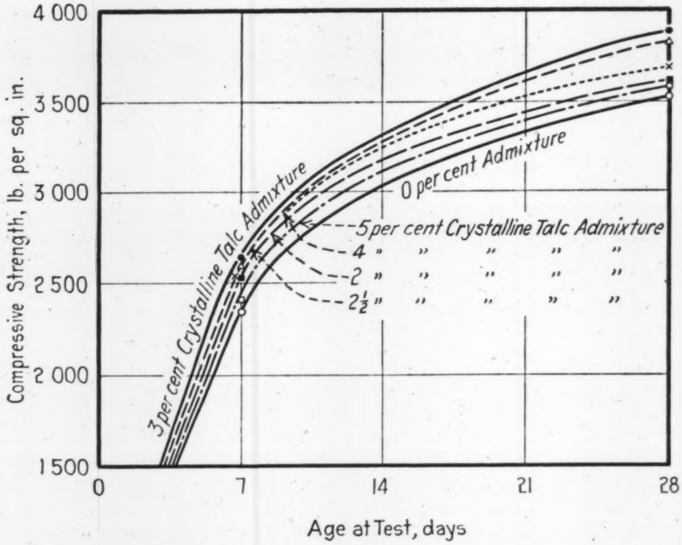

FIG. 3.—Showing Effect of Varying Proportions of Crystalline Talc upon the Compressive Strength of Concrete Mortar.

**Average of nine 2 by 4-in. cylinders each. 1:3 mortar mix. Uniform water-cement ratio.**

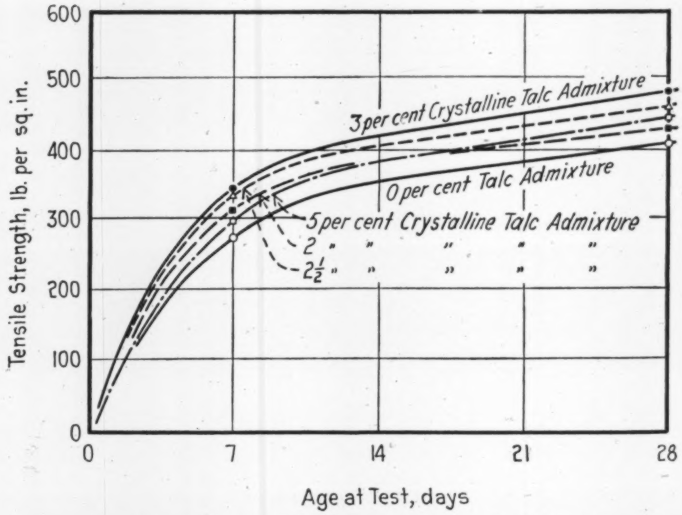

FIG. 4.—Showing Effect of Varying Proportions of Crystalline Talc upon the Tensile Strength of Concrete Mortar.

Average of nine specimens each. **1:3 mortar mix. Uniform water-cement ratio.**

customary in practice, and the admixtures were mixed with the cement dry before adding the aggregates and water, which the author believes to be detrimental rather than otherwise. Experiences of others may differ in this latter respect when dealing with relatively water-absorbing materials, but so far as talc admixture has been studied the indications seem definitely toward lower early strength and less satisfactory lubricating effect if the talc be intermixed with the cement or interground with the cement clinker than may be expected by the mixing with other ingredients at the time the water is added to the mix.

In the case of crystalline talc, there appears to be a more or less critical point at about 2½ to 3 per cent of the weight of the cement where this admixture does the best work with the least amount of water in ordinary 1:6 mixes. Proportions higher than 3 per cent may be used with safety to accomplish especially dense work or for extra good filling of intricate forms, but when these higher percentages are used extra mixing water is required because of the increased volume of the paste. In Figs. 3 and 4, this is graphically shown with reference to 1:3 mortar mixes, with various proportions of crystalline talc as compared with plain concrete. It will be noted that there is a remarkable uniformity in the results shown by tension tests and the results of compression tests. In each case the mortar containing talc in the proportion of 3 per cent of the weight of the cement shows the highest strength and plain concrete the lowest, while the mortars containing 5 per cent talc and 2 per cent talc were about alike.

*Normal Consistency Tests for Admixtures:*

It has been held that the normal consistency test could be used to determine the proportion of extra water that would be needed in concrete containing powdered admixtures.[1] Such a test is found to be reasonably accurate for determining the relative proportions required by varying amounts of any one admixture, but where there is a wide difference in the leading characteristics of admixtures in general, it is believed that the normal consistency test is likely to produce erroneous results and cannot be used as a guide to determine the amount of water to be used in the mix.

Normal consistency takes into consideration the absorbed water as well as water adsorbed or held within the powdered mass by capillarity. When such a powder is distributed through the mix and thus becomes only one part in about 250 parts of the whole, as it does in

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[1] D. A. Abrams, "Tests of Powdered Admixtures in Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part II, p. 618 (1929).

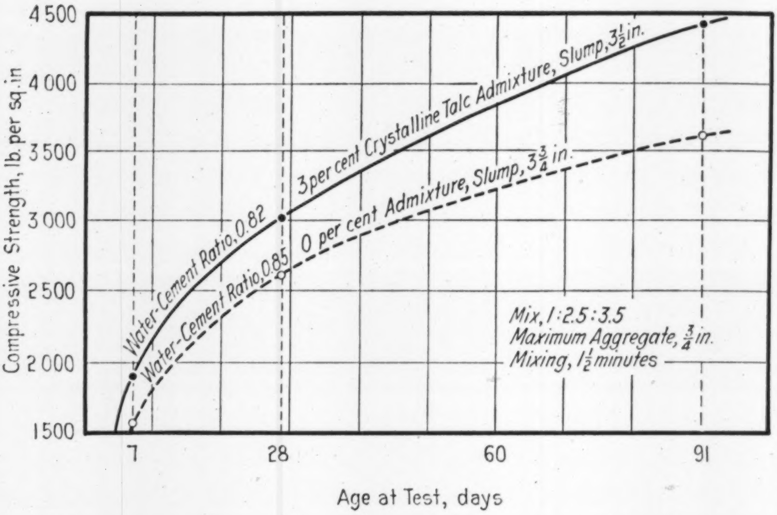

FIG. 5.—Showing Results Obtained on Building Construction (Floor Slabs and Beams). Quantity of Water Used Governed by Workability Desired.

Results shown are average of three cylinders each. Work done by Los Angeles Bureau of Water and Power. Cylinders made and tested by Osborne Laboratories, Los Angeles.

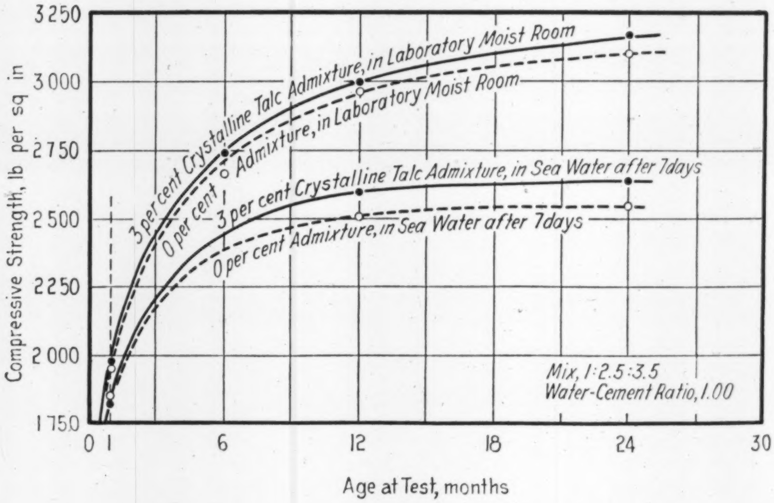

FIG. 6.—Showing Relative Compressive Strength of Concrete in Moist Room Storage and in Sea Water, both With and Without Crystalline Talc Admixture.

Local cement; washed sand; graded aggregate ¾ to 2 in. Each result shown is average of three tests. Cylinders, storage and tests by Osborne Laboratories, Los Angeles.

the usual mix, then the normal consistency of the powder has little if any bearing upon the amount of water needed to satisfy the mix.

This conclusion was reached in the comparison of various cements,[1] as extensive tests showed that the normal consistency of cements was not an accurate guide as to the flow of concrete, and "no definite relation exists between the normal consistency of a cement and the strength of concrete made from that cement."

In a later work by other authors,[2] the question of normal consistency with reference to cements is stated as follows: "It is the opinion of the authors that the determination of normal consistency is of doubtful value for indicating the quantity of mixing water to be used." The same situation seems to apply to the comparison of admixtures with cement and also in comparing one admixture with another.

### *Workability and Uniformity of Concrete Containing Crystalline Talc:*

Workability and uniformity of concrete cannot be determined satisfactorily in laboratory tests and can scarcely be shown in data form except in terms of strength. However, the following quotation from a current publication may be cited as a typical example of the type of work accomplished:[3]

> Concrete was mixed with the smallest practical water content, which was gaged by a Crosby water meter. A slump of about 3½ in. being allowed. A 3 per cent admixture of talc was permitted to improve workability. Some idea of the success which attended the placing of concrete may be obtained from the statement of the engineers that 7500 cu. yd. of concrete were poured in the structure without a single rock pocket being detected.

Figure 5 shows the relative strength of concrete developed on another construction job by mixes with and without crystalline talc, wherein the proportion of water was gaged by the workability desired.

### *Effect of Magnesia Contained in Crystalline Talc:*

A test for proportion of soluble magnesia in concrete with and without crystalline talc admixture was made annually on concrete which has now attained the age of 4½ years, and the results have shown no difference in this respect between the two types of concrete. The test series was conducted to determine whether any portion of the magnesium silicate in talc might become altered to magnesium carbonate or otherwise, and it is clearly shown that no such alteration takes place.

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[1] Report of Subcommittee V on Normal Consistency of Committee C-1 on Cement, *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 252 (1928).

[2] C. H. Scholer and L. H. Koenitzer, "A Study of Fourteen Brands of Standard Portland Cements and Four Early-Strength Cements," *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part II, p. 581 (1930).

[3] "Los Angeles County Bridge Building," Editorial, *Southwest Builder*, May 15, 1931, p. 52.

In this connection it was of interest to know the effect of sea water upon concrete containing crystalline talc. Figure 6 shows the results of an interesting series of tests made for this purpose, made principally for the information of manufacturers of concrete piles, which accounts for the methods used. It seems very evident that relatively early exposure to sea water prevents the development of normal strength in concrete.

The concrete containing crystalline talc showed somewhat higher strength at all ages tested except at 28 days, where the plain concrete in sea water was somewhat stronger than concrete containing talc.

In this test each cylinder was individually mixed by hand. They were all kept in moist storage for the first week after which some of the cylinders were taken to San Pedro Harbor and stored in suitable, open boxes in sea water.

## CONCLUSIONS

It is the conclusion of the author, after analyzing the results of several hundred tests in various laboratories and after rather extensive experience in the conduct of field tests on numerous admixtures, that the utility of any admixture must be judged by giving due consideration to both laboratory data and the actual structural results.

Powdered crystalline talc is a type of material which has natural properties consistent with those requirements which seem theoretically essential for admixture purposes. Our work with crystalline talc is yet far from being as complete as could be desired, but during the past six years the laboratory tests and construction experience with this material have shown a very definite tendency toward improvement in structural qualities and appearance of concrete through the use of crystalline talc as an admixture.

# DISCUSSION

MR. G. A. SMITH[1] (*presented in written form*).—I have reviewed Mr. Wicks' paper with considerable interest because of my particular interest in the subject of the improvement of workability and the effect admixtures and variations of concrete mixtures have in changing the workability of the concrete. The results of the strength tests are very interesting and the consistent trends of the results indicate careful operation and good technique in conducting the tests. However, would not the small changes in strength be a logical consequence, since a quantity of dry material was added to the mix and required a certain proportion of the original mixing water to wet the surface? It has been my observation that additions of any dry material of mineral nature require more water to maintain the same consistency.

It seems to me that Mr. Wicks' paper is somewhat lacking in data which would bear evidence to the fact that crystalline talc is beneficial from the standpoint of improving the workability of the concrete.

It has been my privilege during the past few years to examine and compare a large number of admixtures from the standpoint of their effects on the workability of the concrete. In 1928 a relatively large group of tests was conducted to study Pacific Coast talc and recently examination was made of an Eastern talc and powdered soapstone. Results of these tests are given in the accompanying Table I. I should point out that a 1:2:4 mix by dry rodded volumes was used throughout the tests, but in the three groups of tests different conditions as to materials, temperature, etc., prevailed so that results are not comparable between groups. The workability index was obtained with the Improved Penetration Apparatus[2] and the consistency or flow was determined using fifteen $\frac{1}{8}$-in. drops of the flow table.

In order to maintain the same flow it was necessary, in all cases where admixtures were added, to increase the mixing water. In group 1 it is to be observed that for both Talc-P and Celite the water-cement ratio increased very uniformly with the change in quantity of admixture used. It will also be observed that with both materials the relative workability increased approximately in proportion to the quantity used. However, it should be pointed out that for an equal amount and at the same flow (90) Celite was from eight to nine times more effective than was Talc-P. The relative effectiveness in this case is based on the ratio of the improvements in workability, which are obtained by subtracting 100 from the relative workabilities. That is that any percentage greater than 100 indicates an improvement in workability. Observations relative to Talc-P hold also for the other two materials considered.

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[1] Hydraulic Cement Engineer, Research and Development Dept., Johns-Manville Corp., Manville, N. J.

[2] G. A. Smith, "The Measurement of Workability of Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 505 (1928.)

TABLE I.—RESULTS OF COMPARATIVE WORKABILITY TESTS OF CELITE AND TALC.

| Group | Admixture Used | | Water-Cement Ratio by Volume | Consistency | | Workability | | Compressive Strength, lb. per sq. in. | |
|---|---|---|---|---|---|---|---|---|---|
| | Kind | Amount[a] | | Flow, per cent | Slump, in. | Index | Relative, per cent[b] | 28 days | 90 days |
| 1 | None | 0 | 0.959 | 61 | 3.8 | 423 | 62 | 2679 | 3554 |
| | | 0 | 0.999 | 89 | 5.7 | 262 | 100 | 2499 | 3293 |
| | | 0 | 1.067 | 119 | 7.6 | 174 | 150 | 2096 | 2862 |
| | Celite | 1 | 1.051 | 90 | 6.1 | 204 | 128 | 2431 | 3155 |
| | | 2 | 1 080 | 89 | 6.3 | 175 | 150 | 2399 | 3277 |
| | | 3 | 1.071 | 62 | 4.1 | 216 | 121 | 2680 | 3627 |
| | | 3 | 1.117 | 90 | 6.0 | 151 | 174 | 2460 | 3404 |
| | | 3 | 1.185 | 118 | 7.3 | 94 | 279 | 2103 | 2913 |
| | | 5 | 1.197 | 90 | 6.0 | 123 | 213 | 2308 | 3363 |
| | Talc-P | 1 | 1.014 | 88 | 6.0 | 255 | 103 | 2426 | 3191 |
| | | 2 | 1.015 | 90 | 6.0 | 247 | 106 | 2446 | 3257 |
| | | 3 | 0.978 | 62 | 4.2 | 364 | 72 | 2597 | 3490 |
| | | 3 | 1.026 | 88 | 5.8 | 240 | 109 | 2299 | 3204 |
| | | 3 | 1.091 | 123 | 7.6 | 174 | 150 | 2092 | 2825 |
| | | 5 | 1.034 | 90 | 5.6 | 226 | 116 | 2405 | 3141 |
| | | 10 | 1.055 | 91 | 6.2 | 195 | 134 | 2447 | 3262 |
| 2 | None | 0 | 0.950 | 88 | ... | 227 | 100 | 2943 | 3770 |
| | Celite | 3 | 1.046 | 88 | ... | 138 | 164 | 2878 | 3560 |
| | Talc-E | 3 | 1.000 | 89 | ... | 222 | 102 | 2874 | 3588 |
| 3 | None | 0 | 0.935 | 89 | 4.9 | 288 | 100 | 3080 | 3750 |
| | Celite | 3 | 1.036 | 91 | 5.2 | 165 | 175 | 3019 | 3840 |
| | Talc-S | 3 | 0.954 | 90 | 4.5 | 269 | 107 | 3205 | 3922 |

NOTE: Talc-P is Pacific Coast talc.
Talc-E is Eastern talc.
Talc-S is powdered soapstone.

[a] Amount of admixture expressed as pounds per sack of cement.

[b] Relative workabilities based on workability of plain concrete at flow of 90 per cent.

On page 537 of the paper, Mr. Wicks indicates that because of the direct relation between water, cement and strength there may be serious doubt as to the advisability of using any water-absorbent material for the production of workability in concrete. It has been my observation that though for a given condition the relation between water and cement does bear a relation to the strength, changes in conditions as to mix, etc., also have their bearing on the strength. With variations in the water-cement ratio - strength relations brought about by variations in the essential ingredients it is not illogical to believe that with an absorbent admixture requiring somewhat additional water to maintain a given consistency, a new and distinct water-cement ratio - strength relation will exist, and that at even a higher water-cement ratio a strength will be developed equal to that with a lower water-cement ratio under parallel conditions of no admixture. As confirmation that this is the case, I would refer to my discussion of Mr. D. A. Abrams' paper in the 1929 *Proceedings* of the Society[1] and to "A Study of the Flow Table and the Slump Test" which appeared in the *Journal* of the American Concrete Institute, January, 1931. In the curves showing the water-cement ratio-strength relation for concrete with and concrete without Celite, the Celite curve is displaced to the right and is above the plain curve by an amount equal to the reduction in strength that might have existed had no compensating effect been introduced by the presence of the Celite. Almost an exact duplication of the curves in the flow table study was obtained in a later study to determine the effect of Celite on the modulus of elasticity and in which all plain specimens averaged 2752 and 3234 lb. per sq. in. and all Celite specimens averaged 2818 and 3211 lb. per sq. in. at 28 and 90 days respectively. Each average was for 45 individual tests. In group 1 of the accompanying Table I, the averages for all specimens are as follows:

| ADMIXTURE USED | AVERAGE COMPRESSIVE STRENGTH, LB. PER SQ. IN. 28 DAYS | 90 DAYS |
|---|---|---|
| None | 2420 | 3240 |
| Celite | 2400 | 3290 |
| Talc-P | 2390 | 3200 |

Even though it is necessary when Celite has been added to a mix to use a modified water-cement ratio in order to secure the same consistency, the data presented and those referred to indicate that there is certainly little reason to question the advisability of using certain absorbent admixtures.

MR. F. R. WICKS.[2]—Mr. Smith mentioned Abrams' report on admixtures. I have been very much interested in Mr. Abrams' work. In the work we have done on talc we do not find very much variance between our own work and that of Abrams. The Abrams' discussion takes into consideration certain quantities of admixture below what we consider a critical point, and also above what we consider a critical point. Abrams' report does not take into consideration any of the crystalline talcs or materials of that nature.

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[1] Discussion by G. A. Smith of paper by D. A. Abrams, "Tests of Powdered Admixtures in Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part II, p. 618 (1929).

[2] Consulting Engineer, Pacific Coast Talc Co., Los Angeles, Calif.

As to the results of using certain water-absorbent materials, it is entirely possible to obtain, and you do actually obtain, considerable improvement in workability with certain water-absorbent materials, of which Celite is one.

However, it is the tendency in the field to use a larger proportion of water, and consequently to produce a higher proportion of voids in concrete when water-absorbent materials are used, which is not true with water-repellant materials, either crystalline talc or artificial water-repellant materials.

Very often when using materials containing finely divided microscopic cells such as the diotomite (a great many varieties of diotomite are used) you do get a more uniform concrete but a more porous concrete, even though the pores are more finely divided. However, it is my experience and the experience, I think, of many engineers, that the more dense and less porous you can make concrete, the better it is going to be and the more permanent it is going to be both as regards ultimate strength and ultimate life. If you can use materials that do not require extra water for mixing you have almost invariably produced a higher-quality concrete.

(*Author's closure by letter*).—My paper was limited to a description of results obtained by the use of crystalline talc as an admixture in concrete. Comparisons with other admixtures were not within the scope of the paper. This subject could best be discussed at length in an entirely separate article.

The data submitted by Mr. Smith appear to represent a series of tests comparing three kinds of concrete, wherein the amount of mixing water was so adjusted in each of the three as to produce concretes having similar flow as measured by the flow table.

On page 542 of my paper mention is made of the generally inconclusive results obtained from flow table tests. My experience has been that two kinds of plain concrete, without admixture, may be compared upon the flow table, or two kinds of concrete containing the same admixture may be so compared, but it is not correct to compare plain concrete with admixture concrete or to judge the relative merits of two admixtures by flow table tests. Results so obtained are of but little value in predicting the action of concrete under actual construction conditions. Segregation taking place upon the flow table will often indicate flow relationships quite different from the comparative flow found in the same mixes in field work.

To a lesser extent the same is true of slump tests. During the progress of a construction job, wherein an admixture is being used, the desired consistency may be accurately maintained by keeping the mix to a certain predetermined slump. If the admixture is changed or omitted, then the standard slump must usually be increased or decreased if the same working conditions are desired.

The net result, therefore, of Mr. Smith's tests appears to be that no direct comparison can be drawn between the several kinds of concrete. The tests do show, however, that in any one kind of concrete any change in proportion of water used has an inverse effect upon the compressive strength of the concrete produced, and a direct effect upon the relative workability—an interesting demonstration of the water-cement ratio law in several kinds of concrete.

There are four kinds of talc produced in the Pacific Coast States: Dark fibrous talc similar to soapstone, a white fibrous talc, a semi-crystalline talc and crystalline talc. Of these, only the crystalline talc has come into general use for admixtures in concrete. Mr. Smith does not make it clear which of these varieties was used in the test work he describes.

In the use of crystalline talc admixture in actual construction work, it is customary to reduce the proportion of mixing water below that usually required for plain concrete, the weight of water omitted being approximately equal to the weight of crystalline talc added. When this is done the desired workability is maintained while the strengths obtained are correspondingly increased.

# THE CONCRETE FLOW TROUGH

By Donald M. Burmister[1]

## Synopsis

The design and control of concrete mixtures serves two very important purposes: (1) to produce a concrete of definite specified strength and (2) to produce a concrete of quite definite flow characteristics. The flow table principle seemed to offer greater possibilities for the development of a more satisfactory apparatus for the testing of the wetness of concrete, although the slump principle seemed to give better indications of workability. The flow trough was developed to confine the flow of a mass of concrete in one direction along the axis of a semi-cylindrical trough under the action of ten 1-in. drops. A modified mortar flow trough was similarly developed for the testing of mortars and stuccos.

An investigation of a series of different concrete mixes was made to show that quite definite information as to the workability of concrete can be obtained by means of the flow trough as well as a measure of the wetness, and that results are reproducible within quite narrow limits. The slump test is used for comparison.

The characteristics of a concrete are reflected in varying degrees by its behavior in any given test. The flow trough test furnishes two figures which are indicative of these characteristics, namely, the flow at ten drops and a plasticity coefficient. The results of the investigation show how these figures as indices of workability can be used as a basis for the comparison of concretes and for the design of concrete mixtures. The flow as a measure of the wetness of concrete and the plasticity coefficient as a measure of its plasticity together define a concrete more completely and make its properties more tangible.

## Introduction

The flow trough has been developed to measure the relative flow of concrete mixtures. It incorporates the advantages of the slump cone and the flow table. Since the flow is all confined to one direction, a large definite, measurable displacement is obtained. The application of the apparatus is not confined, however, to checking the wetness of concrete but is useful for obtaining more definite information concerning the workability of concrete.

The properties of concrete are reflected in varying degrees in its behavior in every test as well as upon the job. Its wet, more-or-less fluid nature imparts flow characteristics. Its semi-plastic nature imparts smoothness and mobility. These qualities together with any tendency toward segregation determine to a large extent the workability of the concrete.

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[1] Instructor in Civil Engineering, Columbia University, New York City.

A method of evaluating relatively the effects of these characteristics is outlined in a simple test by means of the flow trough and expressed as the "plasticity coefficient," which is suggested as an index of relative workability of concrete. The plasticity coefficient is indicative of the properties and the behavior of concrete, and provides a more definite basis for judging the workability of concrete and for designing or adjusting the concrete mix to meet field conditions.

A modified form of the flow trough is useful in designing and checking the wetness and workability of mortars and stuccos and is useful in determining the standard consistency of mortars and the normal consistency of neat cement.

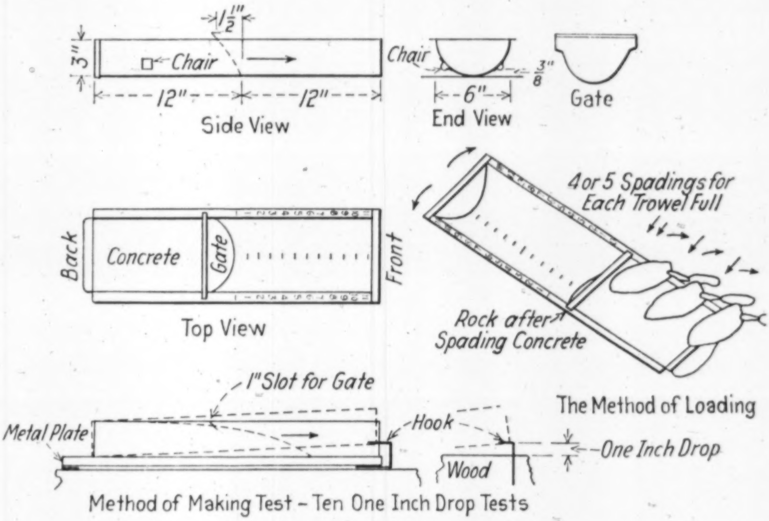
1½"
3"
Chair
12"
12"
Side View
Chair
3/8"
6"
End View
Gate
Back
Concrete
Gate
Front
Top View
4 or 5 Spadings for Each Trowel Full
Rock after Spading Concrete
The Method of Loading
1" Slot for Gate
Metal Plate
Hook
Wood
One Inch Drop
Method of Making Test – Ten One Inch Drop Tests

FIG. 1.—Concrete Flow Trough.

## DESCRIPTION OF THE FLOW TROUGH

The concrete flow trough is illustrated in Figs. 1 and 2. The apparatus consists of a semi-cylindrical trough made of No. 16 gage galvanized iron. The sides are bent over as shown to stiffen the trough and to form side rails on which graduations are stamped. The trough is 6 in. in diameter, 3 in. deep and 24 in. long. The back is closed and the front is left open. A gate made of No. 16 gage galvanized iron, located in the middle of the trough, retains the concrete to be tested. This gate is given a backward slope of 3 in. on 1½ in. in order to obtain a truer flow and to prevent the front of the mass from crumbling. The bottom of the trough and the side rails in front of the gate are graduated in inches for a length of 12 in. Two metal chairs are soldered on the outside of the trough as shown with clearances of $\frac{3}{8}$ in., which permit rocking the trough through an angle of about 10 deg. for the purpose of consolidating the concrete for the test.

## OUTLINE OF TEST PROCEDURE

*Loading the Flow Trough:*

The trough is moistened with a piece of damp waste and the gate inserted in the slots provided. The gate when held by the left hand serves as a handle while loading the trough. Concrete is placed in the trough a trowel-ful at a time, dumped alternately from opposite sides of the trowel to prevent the accumulation of coarse aggregate along one side of the trough. Each trowel-ful is spread longitudinally along the trough from the gate toward the back with the point of the trowel. A blunt nose trowel with a 2-in. wide point is best for this purpose. Each trowel-ful is spaded four or five times, giving two or three of these spadings next to the gate, so that the front of the mass may be uniform.

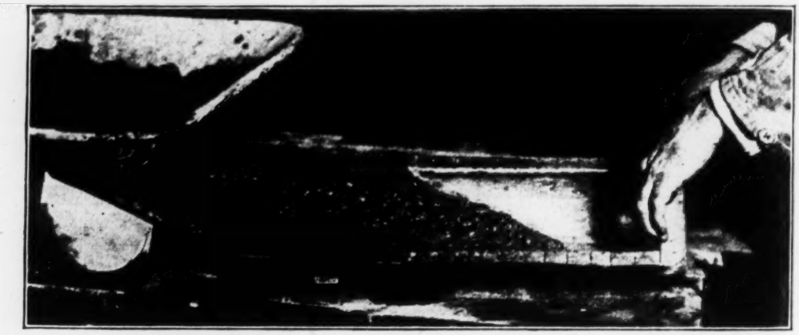

FIG. 2.—Showing the Method of Making the Flow Test.

The trough is then rocked rapidly back and forth with the left hand. It is important that the amount and vigor of rocking given to each trowel-ful be varied according to the wetness of the mix so that the fresh concrete runs together, shows a wet surface, and quakes. This is quite a definite point. A dry mix will require hard rocking, while a sloppy one will take very little if any. The object of the rocking is to make the concrete a homogeneous mass, but not to cause coarse aggregate to settle and compact. More working than is necessary to form a homogeneous mass detracts from the value of the test.

It is important that the concrete be taken from all parts of the mixing pan, so that a representative sample is obtained for test. An accumulation of coarse particles at the back of the trough does not affect results materially. When the trough is full, the top is troweled level with the side rails with as few smoothing motions as possible. The finishing qualities of the concrete are brought out in this operation. If the trough is too full, a small portion of the concrete is removed. The appearance of the back of this confined concrete gives an indication of tendencies toward honeycombing, where coarse aggregate tends to collect against forms.

*The Test Set-up:*

The actual flow determination is made by placing the flow trough on a hard level surface, preferably a metal plate. Under one end of the plate a hook is inserted so as to provide clearance for 1-in. drop, see Fig. 1; or a hook is screwed into a wood surface. The open end of the trough is slid under this hook. The operator stands in front of and faces the open end of the trough.

*The Critical Point:*

Just before pulling the gate, any mortar that may have worked under the gate is wiped away with a piece of damp waste. The gate is pulled by first rotating it out of one slot, when the appearance of the toe of the concrete mass may be noted. If it is honeycombed, the mix is too dry and there is defficiency of mortar.

If, however, mortar drains or leaks from the face of the concrete it indicates that surface tension or capillarity has been broken down by an excess of mixing water, so that the mortar loses to a large extent its binding properties and the ability of retaining all of the mixing water. The mass gives up its water readily with very little working and on standing. This is the critical point for a particular mix. The workability of a concrete can be increased by the addition of water only up to this point, beyond this point segregation and harshness are much more pronounced. It is important that the occurrence of any leakage be noted as it indicates the limit of workability.

*The Flow Determination:*

The flow determination made in the following way gives consistent results. The test is made as soon as possible after the gate is pulled, waiting only long enough for any leakage flow to come to rest and be noted. The test is made by raising the front end of the trough 1 in., the hook serving as a stop, and allowing it to drop under its own weight. This is repeated ten times in ten seconds, causing the concrete to slump and at the same time to flow outward along the trough.

## TABLE I.—DATA OF FLOW AND SLUMP TESTS.

Proportions: on a dry loose basis, reduced to proportions by weight.

| Mix by Volume | Weight of Cement, lb. | Weight of Sand, lb. | Weight of Gravel, lb. |
|---|---|---|---|
| 1:3:6 | 2.70 | ... | .... |
| 1:2½:5 | 3.25 | ... | .... |
| 1:2:4 | 4.05 | 8.4 | 18.0 |
| 1:1½:3 | 5.40 | ... | .... |
| 1:1:2 | 8.10 | ... | .... |

Cement: Atlas Portland cement, from Northampton, Pa. Unit weight 94 lb. per cu. ft.

Sand: Series 1. A Long Island sand having a fineness modulus of 2.49 and weighing 98 lb. per cu. ft. dry and loose.

Series 2. A Long Island sand having a fineness modulus of 2.69 and weighing 98 lb. per cu. ft. dry and loose.

Gravel: A Long Island gravel regraded to a fineness modulus of 6.96 and weighing 104 lb. per cu. ft. dry and loose.

A flow test was made upon each batch and immediately thereafter a slump test was made after thoroughly remixing the batch.

| Mix | Mixing Water, cc. | Water-Cement Ratio | Slump, in. | Leakage Flow, in. | Flow at Five Drops, in. | Flow at Ten Drops, in. | Plasticity Coefficient |
|---|---|---|---|---|---|---|---|
| SERIES 1.—FINENESS MODULUS OF SAND 2.49 | | | | | | | |
| 1:3:6 | 1050 | 1.28 | ½ | 0 | 1¼ | 3 | 0.83 |
| | 1100 | 1.34 | 1½ | 0 | 2 | 4½ | 0.89 |
| | 1150 | 1.41 | 3 | ½ | 3 | 6 | 0.92 |
| | 1200 | 1.47 | 5 | 1 | 4¼ | 7 | 1.05 |
| | 1250 | 1.54 | 7 | 2 | 7 | 12 | 1.00 |
| 1:2½:5 | 1100 | 1.12 | 2 | 0 | 2 | 4 | 1.00 |
| | 1150 | 1.17 | 3½ | 0 | 3 | 6 | 1.00 |
| | 1200 | 1.22 | 5 | ½ | 4 | 7 | 1.07 |
| | 1250 | 1.27 | 7 | 1½ | 6½ | 10 | 1.15 |
| | 1300 | 1.32 | .. | 2 | 7 | 12 | 1.00 |
| 1:2:4 | 1050 | 0.86 | 1 | 0 | 1½ | 3 | 1.00 |
| | 1100 | 0.90 | 3 | 0 | 2¼ | 4½ | 1.00 |
| | 1150 | 0.94 | 4½ | 0 | 3¼ | 6 | 1.09 |
| | 1200 | 0.98 | 6½ | 0 | 4¾ | 8 | 1.19 |
| | 1250 | 1.02 | .. | 0 | 6¼ | 10 | 1.25 |
| 1:1½:3 | 1100 | 0.68 | ½ | 0 | 1¼ | 2½ | 1.00 |
| | 1150 | 0.71 | 2½ | 0 | 2½ | 4½ | 1.11 |
| | 1200 | 0.73 | 4½ | 0 | 3½ | 6 | 1.17 |
| | 1250 | 0.77 | 6½ | 0 | 5 | 8 | 1.25 |
| | 1300 | 0.80 | .. | 0 | 6 | 9 | 1.33 |
| | 1350 | 0.83 | .. | 0 | 7 | 10 | 1.40 |
| 1:1:2 | 1350 | 0.55 | ½ | 0 | 1 | 2 | 1.00 |
| | 1400 | 0.57 | 1 | 0 | 1¾ | 3 | 1.17 |
| | 1450 | 0.59 | 3 | 0 | 2½ | 4 | 1.25 |
| | 1500 | 0.61 | 5 | 0 | 3¾ | 5½ | 1.36 |
| | 1550 | 0.63 | 7 | 0 | 4¾ | 6½ | 1.46 |
| | 1600 | 0.65 | .. | 0 | 5½ | 8 | 1.37 |
| | 1650 | 0.67 | .. | 0 | 7 | 11 | 1.27 |

TABLE I—*Continued.*

| Mix | Mixing Water, cc. | Water-Cement Ratio | Slump, in. | Leakage Flow, in. | Flow at Five Drops, in. | Flow at Ten Drops, in. | Plasticity Coefficient |
|---|---|---|---|---|---|---|---|
| SERIES 2.—FINENESS MODULUS OF SAND 2.69 | | | | | | | |
| 1:3:6 | 950 | 1.16 | 1 | 0 | 2 | $4\frac{1}{2}$ | 0.89 |
| | 1025 | 1.26 | 4 | $\frac{3}{4}$ | $3\frac{3}{4}$ | 7 | 0.96 |
| | 1100 | 1.35 | $6\frac{1}{2}$ | $1\frac{1}{2}$ | 6 | 9 | 1.16 |
| | 1175 | 1.44 | .. | $3\frac{3}{4}$ | $7\frac{1}{2}$ | 11 | 1 02 |
| $1:2\frac{1}{2}:5$ | 1000 | 1.02 | 1 | 0 | 2 | 4 | 1.00 |
| | 1075 | 1.09 | 4 | 0 | 4 | $7\frac{1}{2}$ | 1.07 |
| | 1150 | 1.17 | $7\frac{1}{2}$ | $1\frac{1}{2}$ | $6\frac{1}{4}$ | $9\frac{1}{4}$ | 1.19 |
| | 1225 | 1.25 | .. | 4 | $8\frac{1}{2}$ | $11\frac{3}{4}$ | 1.10 |
| 1:2:4 | 1025 | 0.84 | 1 | 0 | 2 | $3\frac{3}{4}$ | 1.07 |
| | 1100 | 0.89 | $3\frac{1}{2}$ | 0 | $3\frac{3}{4}$ | $6\frac{3}{4}$ | 1.11 |
| | 1175 | 0.95 | $6\frac{1}{2}$ | 0 | $5\frac{1}{2}$ | $8\frac{1}{2}$ | 1.29 |
| | 1250 | 1.02 | .. | $\frac{1}{2}$ | $6\frac{1}{2}$ | $9\frac{1}{2}$ | 1.31 |
| | 1325 | 1.08 | .. | 2 | 8 | 11 | 1.28 |
| $1:1\frac{1}{2}:3$ | 1125 | 0.655 | 1 | 0 | $1\frac{1}{2}$ | $2\frac{3}{4}$ | 1.09 |
| | 1175 | 0.68 | 3 | 0 | $2\frac{1}{2}$ | $4\frac{1}{4}$ | 1.18 |
| | 1250 | 0.725 | 5 | 0 | $3\frac{1}{2}$ | $5\frac{1}{2}$ | 1.27 |
| | 1325 | 0.77 | 7 | 0 | 5 | $7\frac{1}{2}$ | 1.33 |
| | 1400 | 0.815 | .. | 0 | 6 | $8\frac{1}{2}$ | 1.41 |
| | 1475 | 0.855 | .. | 0 | $7\frac{1}{2}$ | $10\frac{1}{2}$ | 1.43 |
| 1:1:2 | 1300 | 0.52 | 1 | 0 | $1\frac{1}{2}$ | $2\frac{1}{2}$ | 1.20 |
| | 1375 | 0.56 | 2 | 0 | 2 | $3\frac{1}{4}$ | 1.24 |
| | 1450 | 0.595 | 4 | 0 | $3\frac{3}{4}$ | $4\frac{1}{1}$ | 1.30 |
| | 1525 | 0 625 | 6 | 0 | $3\frac{3}{4}$ | $6\frac{1}{4}$ | 1.20 |
| | 1600 | 0.655 | 8 | 0 | $5\frac{3}{4}$ | $8\frac{1}{2}$ | 1.35 |
| | 1675 | 0.685 | .. | 0 | 7 | $9\frac{3}{4}$ | 1.44 |
| | 1750 | 0.715 | .. | 0 | $8\frac{1}{2}$ | $11\frac{1}{2}$ | 1.48 |

It is important to raise the flow trough until it touches the hook but without jarring the concrete, and to allow the tips of the fingers to slide off of the side rails, thus letting the trough drop freely. This operation should not be hurried nor the fingers jerked away. The number of inches the concrete has flowed along the trough is a measure of its wetness, and is estimated to the nearest $\frac{1}{4}$ in.

The average of two or more tests is desirable. Two flow trough determinations can be made in about the same time required for one slump test. Two determinations can be made upon the same batch of concrete with very little drying or stiffening of the concrete and they will check within quite narrow limits.

The test is primarily a measure of the work required to cause flow in concrete. As the flow is all confined to one direction, a large definite measurable displacement is obtained.

## DISCUSSION OF TEST RESULTS

A comparative series of tests was made on various concrete mixes using the flow trough and the slump cone to test the reliability of the flow trough as a method of measuring wetness of concrete and of obtaining more definite information as to the properties of concrete. The proportions were those which would bring out differences in workability, the cement content only of the mix being varied. The water content was varied from batch to batch by small increments over a wide range. This method was adopted in order to obtain

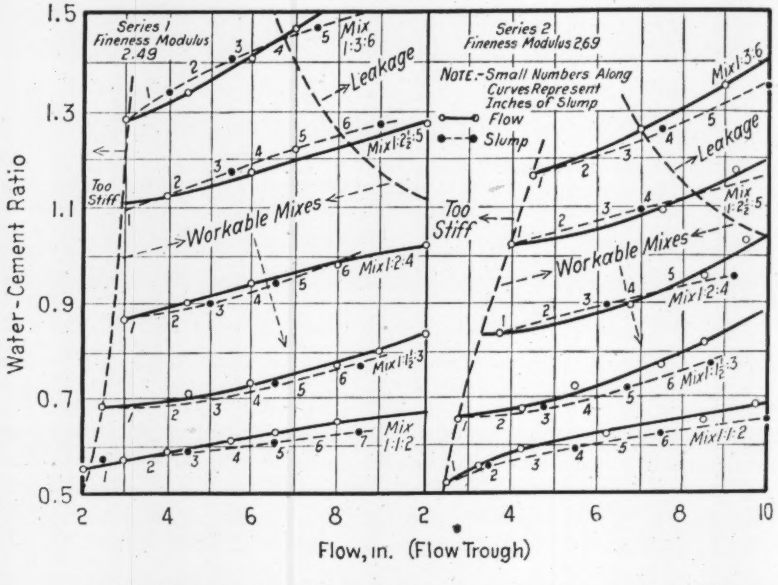

FIG. 3.—Showing the Relation Between the Flow at Ten Drops and the Slump, for Different Concretes.

average curves of the variation of the different factors with the water content over a wide range rather than by using duplicate tests for only a few mixes.

The test results are given in Table I for two series of concretes. The data for series 1 and 2 have been plotted in Fig. 3 with water-cement ratio as ordinates and the flow at ten drops obtained by the flow trough as abscissas. The corresponding slumps obtained by the slump cone are shown by the dotted curves, the figures thereon indicating the corresponding slump in inches.

When the flow is less than two or three inches there is practically no quake and such mixes in the ordinary sense are not classified as workable. The limiting curve for workability is shown where the leakage flow occurs. When the slump curve is above the flow curve, the mix must be wetter to produce the slump indicated than for those mixes where the slump curve is below the flow curve. This is likewise shown by the zero point for the slump curves. Note also the effect of grading of the sand on the relative positions of the two sets of curves.

The flow test gives satisfactory results over a very wide range; and with flow confined to one direction these results are quite consistent for those harsh lean concretes where the slump test is unsatisfactory and indeterminate.

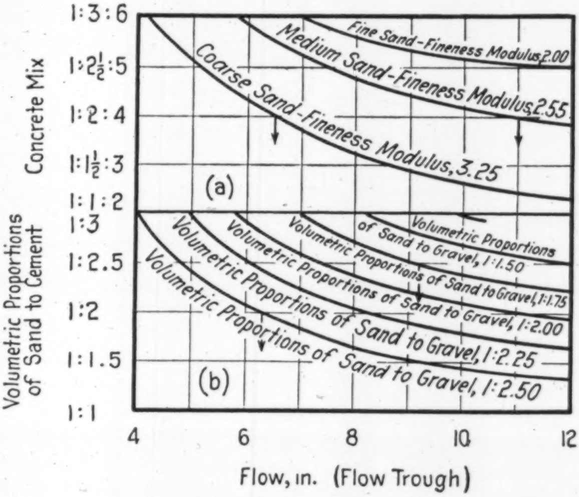

FIG. 4.—Showing the Effect of Grading and Proportions of the Aggregates on the Critical Point. No leakage flow indicated to the flows given for these curves.

(*a*) Shows the effect of grading the sand.
(*b*) Shows the effect of proportions of sand and gravel.

In Fig. 4, curves are plotted showing the wettest mix that can be used with sands of different gradings and with different proportions of sand to gravel before a leakage flow occurs. These curves are obtained from data not included in the results tabulated. In general, the finer the sand and the greater the proportion of sand in the mix the less is the tendency for all mixes to segregate and the greater is the workability. The maximum flows without leakage are indicated for the following:

| Mix | Fineness Modulus | Proportion of Sand to Coarse Aggregate | Maximum Flow Without Leakage, in. |
|---|---|---|---|
| 1:2:4 | 3.25 | ........ | 6½ |
| 1:2:4 | 2.55 | ........ | 11 |
| 1:2:5 | .... | 1:2.5 | 6¼ |
| 1:2.5:4.4 | .... | 1:1.75 | 9¼ |

## THE PLASTICITY COEFFICIENT

The properties of concrete are reflected in varying degrees in its behavior. A concrete may be more plastic and less wet, or it may be wetter and less plastic. Two such concretes are quite different in their properties and behavior and their workabilities are judged accordingly.

It was observed that the flows for successive drops were not proportional and equal for all concretes as might have been expected for the same degree of wetness. To investigate this, the test was divided into two parts and the flows determined at both five and ten drops in order to differentiate between the qualities of a concrete producing these effects. The concrete in the flow trough appears to function somewhat differently in the two halves of the test. In order to compare flows at the two stages of the test, twice the flow at five drops was used. These results are plotted in Fig. 5. (Compare with Fig. 3.) It is observed that by plotting twice the flow at five drops the flow and slump curves more nearly coincide for the more workable concretes in which a true slump can be obtained. Furthermore the average value for the zero point of the slump curves is now very nearly 2. The equivalent slump in inches may therefore be taken approximately equal to twice the flow at five drops, minus 2. These results would indicate that the flow at five drops and the slump test measure the same property.[1] From the above relationship for equivalent slump, the flow can be correlated with standard practice and with specifications.

When the flows at five and ten drops were compared, a consistent variation, not only with the richness of the mix, but also with the increasing water content of the mix developed. In order to evaluate this relationship the ratio:

$$\frac{\text{Twice the Flow at Five Drops}}{\text{Flow at Ten Drops}}$$

which is called the plasticity coefficient, was determined as a relative measure of the qualities of concrete producing the observed effects.

---

[1] See the author's Discussion of the Slump and Flow Tests, *Journal*, Am. Concrete Inst., May, 1931, p. 1151.

This plasticity coefficient is suggested as an index of relative workability for any given concrete. The determination requires that the flows at both five and ten drops be carefully noted to the nearest ¼ in.

When leakage occurs and is well defined, that is, greater than one inch, the mix can no longer be classified as a uniform concrete. For such concretes there is no definite plasticity coefficient, but for comparison it may be determined by subtracting the leakage flow from twice the flow at five drops, for the ratio.

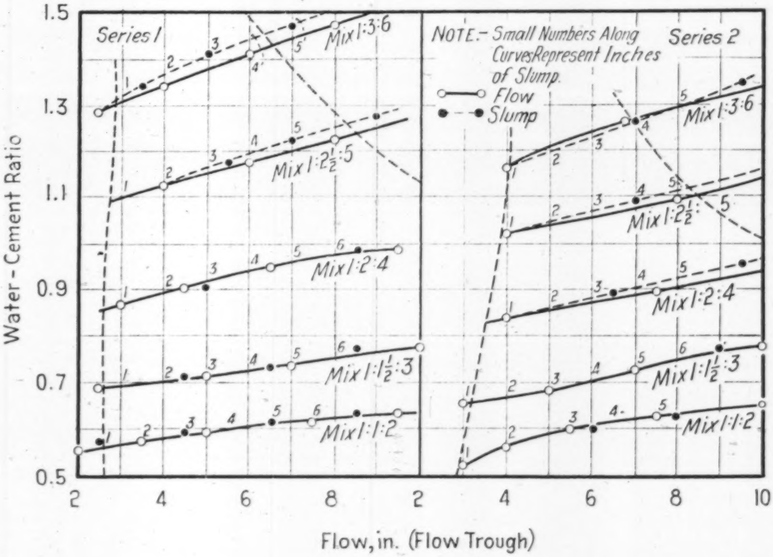

FIG. 5.—Showing the Relation Between the Flow at Five Drops and the Slump, for Different Concretes.

In Fig. 6, curves are plotted on the left showing the variation of the plasticity coefficient with flow within the workable range for concretes of series 1 and 2. The plasticity coefficient increases both with the richness of the mix and with the increase in water content, which agrees with the order of their workability. Judging from the handling and placing properties, a mix cannot be called workable unless the plasticity coefficient is greater than 1.10 with a flow greater than 2 in. but with no evidence of any leakage.

The curves in Fig. 6 are replotted on the right to show in a more convenient way the variation of the plasticity coefficient with the richness and the wetness of the mix.

## DISCUSSION OF THE FUNCTIONING OF THE FLOW TROUGH

The difference in the behavior and properties of concretes is largely due to the effect of fine material in the mix, whether it be cement, admixtures, or silt and clay in the aggregates. When present in sufficient quantities, this material acts as a lubricant and, more important still, it tends through its effect upon surface tension or capillarity to retain all of the mixing water within the mass, producing a plasticity which varies with the amount and kind of this fine material and also with the amount of mixing water.

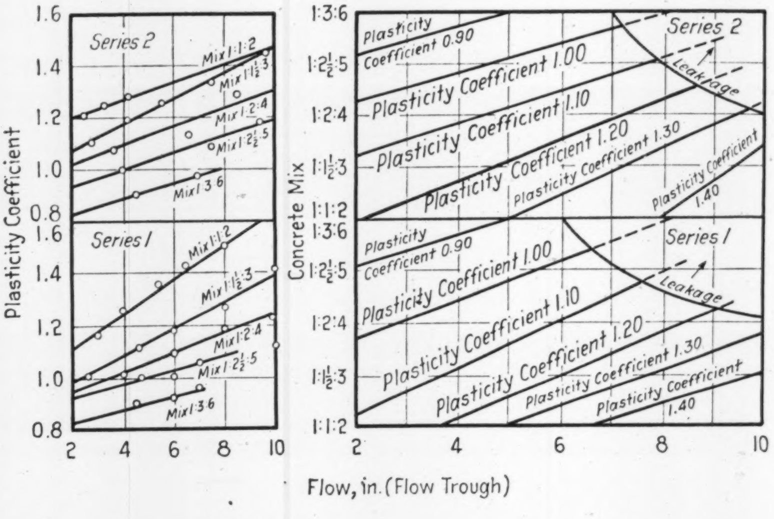

FIG. 6.—Showing the Effect of the Richness and Wetness of Concrete on the Plasticity Coefficient.

In a plastic concrete, the particles of coarse aggregate, when separated by sufficient plastic mortar, flow more smoothly over one another, because of a low shearing resistance, than in a non-plastic or harsh one, consequently the mass as a whole deforms more readily. Under the action of the drop tests, the plastic concrete at the beginning will slump more and flow farther for the same degree of wetness. The confining of the flow to one direction increases this effect.

For the non-plastic or harsh concretes, the flow is retarded in the first stage because of a lack of sufficient plastic mortar, which results in a greater shearing resistance. The test results show a large enough consistent difference, so that the flow at five drops may be taken as a measure of the effect of varying degrees of plasticity in concrete.

The flow for successive drops in the last half of the test, after the concrete has assumed quite a uniform slope from back to front, is more nearly directly proportional to the applied work and the increase in wetness of the concrete. The total flow at ten drops, however, is taken as the measure of the wetness of the concrete, because it is quite impossible to differentiate absolutely between the effects of wetness and plasticity.

TABLE II.—DATA OF COMPARATIVE WORKABILITY TESTS.

**Proportions: dry loose volumes of the following:**

**1:2½:5 plain concrete, proportion of sand to gravel 1:2.**
**1:2½:4 plain concrete, proportion of sand to gravel 1:1.6.**
**1:2½:5 concrete with 3 per cent diatomaceous earth admixture by weight of cement.**

Sand: Long Island sand having a fineness modulus of 2.80 and weighing 98 lb. per cu. ft.
Gravel: Long Island gravel having a fineness modulus of 7.20 and weighing 104 lb. per cu ft.

Three flow tests were made upon each batch and two batches were tested for each increment of water added. The average flows at five and ten drops are tabulated.

| Mix | Mixing Water, cc. | Water-Cement Ratio | Leakage Flow, in. | Flow at Five Drops, in. | Flow at Ten Drops, in. | Plasticity Coefficient |
|---|---|---|---|---|---|---|
| 1:2½:5 plain | 950 | 0.97 | 0 | 2.00 | 4.00 | 1.00 |
| | 1000 | 1.02 | 0 | 2.75 | 5.30 | 1.04 |
| | 1050 | 1.07 | 0 | 3.48 | 6.50 | 1.08 |
| | 1100 | 1.12 | 0 | 4.20 | 7.50 | 1.12 |
| | 1150 | 1.17 | 1 | 5.25 | 8.75 | 1.08 |
| 1:2½:5, 3 per cent Celite | 1000 | 1.02 | 0 | 1.50 | 2.75 | 1.09 |
| | 1050 | 1.07 | 0 | 2.25 | 4.00 | 1.13 |
| | 1100 | 1.12 | 0 | 2.87 | 5.00 | 1.15 |
| | 1150 | 1.17 | 0 | 3.82 | 6.50 | 1.18 |
| | 1200 | 1.22 | 0 | 4.35 | 7.25 | 1.20 |
| | 1250 | 1.27 | 0 | 5.00 | 8.25 | 1.21 |
| 1:2½:4 | 900 | 0.92 | 0 | 2.00 | 3.75 | 1.07 |
| | 950 | 0.97 | 0 | 3.25 | 5.80 | 1.12 |
| | 1000 | 1.02 | 0 | 3.95 | 6.75 | 1.17 |
| | 1050 | 1.07 | 0 | 4.83 | 8.05 | 1.20 |
| | 1100 | 1.12 | 0 | 5.55 | 8.90 | 1.25 |

Therefore the flow for the more plastic concretes is always proportionately greater for the first half of the test than for the less plastic ones, for which the flow at all stages is very nearly proportional to the work done.

## BASIS OF COMPARISON AND DESIGN OF CONCRETE MIXTURES

The purpose of such an index as the plasticity coefficient is to provide a figure or index which will be indicative of the properties of concrete and one which may be used as the basis for the comparison of concretes and for the design of concrete mixtures. The properties and the behavior of any concrete are made more tangible by means of the criteria, plasticity coefficient and flow, which, when taken together, define more completely the relative workability of any concrete.

There are two bases of comparison of concretes within the workable range, as limited by the occurrence of a leakage flow, that are quite definite and simple of application, namely, (1) the comparison of plasticity coefficients for equal flows; and (2) the comparison of flows for equal plasticity coefficients. A high plasticity coefficient as a criterion of the desirable qualities of concrete is indicative of a uniform and homogeneous product with little tendency toward segregation. The flow and the plasticity coefficient together are indicative of the relative ease with which a concrete may be handled and placed.

On the other hand, a low plasticity coefficient indicates a lean watery concrete which is harsh and will tend to segregate badly with the formation of honeycombed surfaces and laitance, or it indicates a very dry or harsh mix, which is difficult to place. Any considerable working to make such concretes flow into forms will only aggravate their undesirable tendencies, especially near the limit of the workable range.

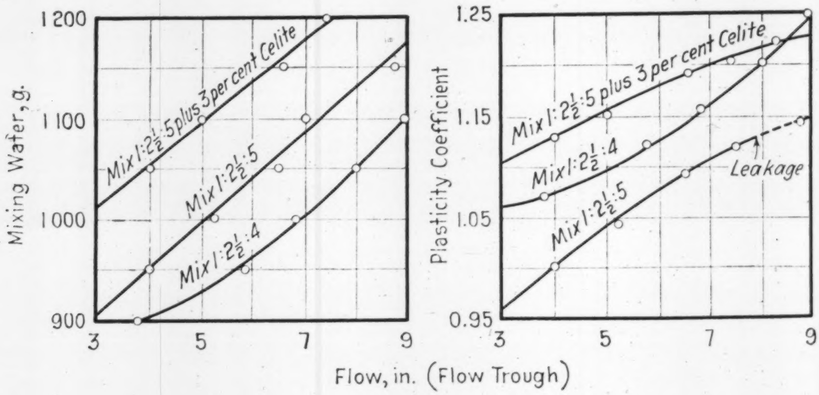

FIG. 7.—Showing the Effects of **Diatomoceous Earth** Admixture and of Proportion of Gravel on Workability of 1:2½:5 Concrete.

As an illustration, a plain 1:2½:5 concrete was first used, second an admixture of 3 per cent diatomaceous earth by weight of cement was added, and third the proportion of gravel was reduced to 1:2½:4 to show how the flow trough can be used to obtain quite definite information by which to judge the relative workability. The data for these tests are given in Table II and the results are shown graphically in Fig. 7.

The relative positions of the curves on the left show the relative amounts of mixing water required by each mix to bring it to any given flow. The relative positions of the curves on the right, using the plasticity coefficient and flow as criteria of workability, are indicative of the properties and the relative workability of the respective mixes. The comparison can be made on either of the two suggested bases.

The general trend of the characteristics of a concrete is of very great importance in judging workability or in selecting the most suitable concrete both as to flow and as to workability.

From the design standpoint there are three simple requirements which can be specified in addition to the water-cement ratio governing the strength of the concrete:

1. No evidence of a leakage flow;
2. A minimum value for the plasticity coefficient consistent with the class of concrete and with field conditions; and
3. A maximum flow suitable for the given field conditions.

In meeting these three requirements a concrete of very definite characteristics, workability, and uniformity may be obtained.

For the design of concrete mixtures, curves similar to those of Figs. 4, 6, and 7 are very useful. For example, assume that a flow of 7 in. were necessary for placing conditions and that a plasticity coefficient of at least 1.10 was specified for the given conditions. The leanest mix from Fig. 6 that could be used for these given materials would be a $1:2\frac{1}{4}:4\frac{1}{2}$ plain concrete. The flow of such a concrete at five drops must be $\frac{7 \times 1.1}{2}$, or slightly greater than $3\frac{3}{4}$ in. The equivalent slump of this concrete would be $7 \times 1.1 - 2$, which is approximately $5\frac{1}{2}$ in.

It must be observed that with this mix the critical point is almost reached and that this mix might tend to segregate badly with much working. Since the critical point is also a very important criterion of workability, a richer mix should be used, such as a 1:2:4 concrete. Or by refering to Figs. 4 and 7 other proportions or gradings of the aggregates may be used, in order to obtain the desired workability.

This basis of design means that to meet all three of these requierments at the same time in a concrete mix is to insure the quality specified, whether the adjustment in the mix is obtained by the use of more cement, admixtures, or by such gradings and proportions of the aggregates as are necessary. This may be accomplished by trial methods, such as are outlined for Fig. 7.

This method of design and comparison has the advantage of providing numerical indices, which make the properties of concrete more tangible and serve as a more definite basis for the design and comparison of concrete mixtures.

## THE MORTAR FLOW TROUGH

The flow trough used for determining the wetness and plasticity of mortars is similar to that used for concrete and embodies all of its advantages. The mortar flow trough is illustrated in Fig. 8. It is a semi-cylindrical trough of No. 16 gage metal 2 in. in diameter, 1 in. deep and 7 in. long with the outer 3 in. of its length graduated in inches and tenths. A support is provided at the back as indicated. A gate inserted in the slots in the middle of the trough retains the mortar to be tested.

The flow trough is moistened with a piece of damp waste and the gate inserted. The trough is then filled one-quarter full at a time. Very dry mortars, such as the standard 1:3 Ottawa sand mortar or similar dry ones, must be well thumbed into place, especially under the gate and at the back, as in making briquets. A few taps on the back of the trough with a trowel will prevent the mass as a whole from sliding instead of flowing. It is essential that the mortar should not slide or break entirely away from the back.

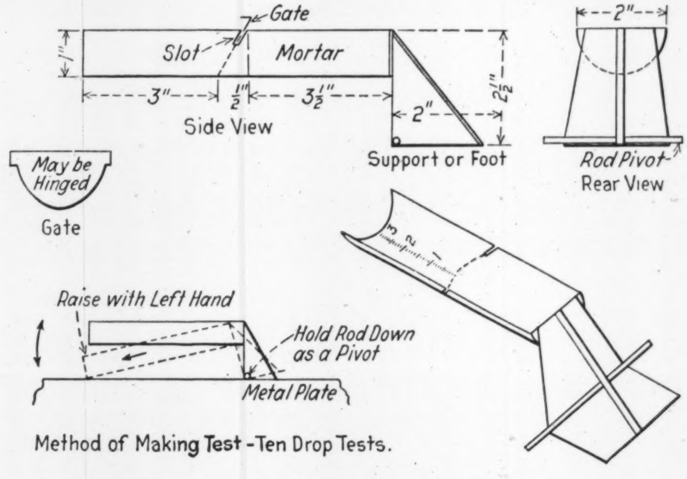
Gate
Slot
Mortar
1"
3"
1/2"
3 1/2"
2 1/2"
2"
Side View
Support or Foot
2"
Rod Pivot
Rear View
May be Hinged
Gate
3
2
1
Raise with Left Hand
Hold Rod Down as a Pivot
Metal Plate
Method of Making Test - Ten Drop Tests.

FIG. 8.—Mortar Flow Trough.

Wetter mixes such as brick mortars should be thumbed only sufficiently to make the surface appear wet. The top surface should then be struck off level with the sides and troweled smooth with as few smoothing motions from front to back as possible. The smoothness of the mortar can be judged at this point.

To make the flow determination, the flow trough filled with mortar is placed on a hard level surface, preferably a metal plate. The front end of the flow trough is held up with the left hand and the gate is carefully pulled by rotating it first out of one slot. In this position the support of the flow trough rests flat on the plate or table, thus bringing the trough to a horizontal position. The ends of the rod, projecting from the support, when held down by the right hand serve as a pivot. The front end of the flow trough is then allowed to drop under its own weight, causing the mortar to flow. The front end is again raised until the support touches the plate lightly and again allowed to fall. This operation is repeated ten times in all in ten seconds, and the final flow is recorded to the nearest 0.1 in.

This flow measures the relative wetness of mortars. As the flow is confined to one direction only, a large definite measurable displacement is obtained. The sensitivity of the flow trough is therefore greater than either the mortar flow table or the slump obtained on a 2 by 4-in. cylinder. A flow of 2½ in. on the flow trough corresponds to a flow of about 25 on the flow table. This flow on the flow table is only ½ in. radially outward and must be estimated for all intermediate values between 0 and 25, which is the usual workable range of mortars.

The flow obtained with the standard 1:3 Ottawa sand mortar mixed to standard consistency by means of the flow trough is ½ in. (plus or minus 0.1 in.). This represents about the driest mix that can be measured accurately by any means. For any other sand this same flow can be obtained easily by trial methods for the purpose of comparing such a sand with the standard Ottawa sand in strength tests.

The flow of a good brick mortar as determined by the flow trough ranges from 2 to 3½ in. (found by proportion in terms of ten drops). The stiffening effect of a mortar on standing can be measured readily so that the size of the batch on the job can be varied accordingly. The effect of lime or other admixtures upon the smoothness, plasticity, and flow of mortars can also be measured.

The flows suitable for prepared stuccos range from 1½ to 2¼ in. This corresponds to a slump of ⅜ to ½ in. measured by means of a 2 by 4-in. cylinder as recommended by the Portland Cement Association for stuccos.

Neat cement mixed to normal consistency gives a flow of 1 in. in the flow trough (plus or minus 0.1 in.).

The advantages of the flow trough are its greater sensitivity, large measurable flow, and indications of smoothness and plasticity. The mortar flow trough is both a laboratory and a field device for designing and controlling the consistency of mortars.

*Acknowledgment.*—The author deeply appreciates the valuable suggestions received during the preparation of this paper from A. H. Beyer, Director of the Civil Engineering Testing Laboratories, Columbia University, New York City.

# DISCUSSION

Mr. F. O. Anderegg[1] (*presented in written form*).—In the test described by Mr. Burmister, the method of packing the concrete in the flow trough has a marked influence on the flow during the first ten drops; but if the dropping is continued a steady flow rate is reached which changes only slightly as the concrete flows along the trough. If the slump is about one inch (normal consistency) the flow rate is steady; for dryer consistencies the rate has an increasing tendency while with wetter mixes the rate decreases slightly.

On experimenting with a large number of gradings at different consistencies it was found that the flow trough was more sensitive for the dryer mixes, while the slump test brought out distinctions better with the wetter ones.

Two factors complicate the flow. One is the packing or dilatancy. If the concrete gets any packing, then before it can assume a steady rate of flow excess work has to be done to lift the packed particles up: that is, the mass has to be dilated. The other factor is the setting of the cement which begins to start almost as soon as water is added. The concrete should be kept stirred to eliminate these complications when making any study among the other factors affecting workability. Papers covering the dilatancy phenomenon include one by Williamson and Heckert[2] and another by C. F. Jenkin.[3]

Aside from these two complicating factors, which can be eliminated, others affect the flowability of concrete (in this discussion it is assumed that no segregation shall occur). The first of these is the amount of lubricating water. When water is added to concrete some is absorbed within the porous aggregates, some is combined with the cement, some wets the surface of the aggregate, some fills the pores and some serves to distend the mass and lubricate the flow of the particles over each other. The outer layers of moisture on the solid particles are more loosely held and have lubricating action. Therefore, the workability can be helped by the introduction of fine material with its increased water envelopes. Colloidal substances such as clay, lime, hydrated cement, etc., are helpful as are finely divided solids like diatomaceous earths and other siliceous admixtures, as are hydrophilic organic substances, for example the carbohydrates.

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[1] Consulting Specialist on Building Materials, Pittsburgh, Pa.

[2] R V. Williamson and W. W Heckert, "Some Properties of Dispersions of the Quicksand Type," *Industrial and Engineering Chemistry*, Vol. 23, No. 6, p. 667, June, 1931.

[3] C. F. Jenkin, "The Pressure Exerted by Granular Material," *Proceedings*, Royal Soc. London, Vol. 131A, p. 53 (1931).

A second factor is the grading of the aggregate. With particles of one size only present considerable work has to be done to dilate the system, but if a variety of sizes is present the smaller ones act as ball bearings. The system that has the maximum workability due to size distribution alone is covered by the formula:

$$P = \frac{r^{\log d} - r^{\log d_1}}{r^{\log d_2} - r^{\log d_1}}$$

where $P$ is the cumulative percentage below any size $d$, and $d_1$ and $d_2$ are the lower and upper size limits of the system, and $r$ is the ratio between successive fractions. With sieves differing in dimension by the factor 2 (standard sieves), $r$ becomes about 1.2 for ordinary aggregates.

A third factor is the deviation of the particles from the spherical shape, while the fourth is the presence of certain hydrophobic organic substances such as paraffin and calcium stearate which have a true lubricating action.

MR. A. D. CONROW[1] (*presented in written form*).—At the present time, the flow test, because of its cumbersomeness, is considered applicable for use only in the laboratory. It has considerable advantage over the slump test in that the personal element is less a factor and probably more consistent duplication of results can be obtained. Mr. Burmister is to be commended in the development of his flow trough because it makes the use of the flow test in the field as convenient as the slump test and so long as we have no better method of indicating the consistency and working qualities of concrete, the flow trough is a distinct advance in the making of such a test. The test should be more sensitive than when made with the flow table; thus finer degrees of measurement should be possible.

The test, however, still has all of the deficiencies of the flow table in that the forces tending to cause flow or deformation diminish with each drop due to a decrease in head, and the amount of decrease in force with each drop will be different with each change in consistency, whether caused by change in water content, cement content, grading of aggregates or combination of these factors. This variation of forces acting in such a test causes a certain set of values such as flow or plasticity coefficient, applicable to concrete for a certain use, to apply only to a certain class of concrete in which certain sizes and nature of aggregate are in use. A different set of values will apply to concrete for the same use when a different type and size gradation of aggregate may be used. For instance when a rounded gravel aggregate is used a flow value of 6 in. and a plasticity coefficient of 1.1 might be found to give a consistency for best conditions of placing in medium reinforced wall construction. If an angular crushed stone coarse aggregate is used, a flow value of 8 in. and plasticity coefficient of 1.15 might be necessary for equal facility in placing, and if a sand mortar with pea gravel is used, probably a flow value of 4 in. would be equally satisfactory.

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[1] Special Engineer, Ash Grove Lime and Portland Cement Co., Chanute, Kans.

This limits the application of the test values to specific materials to be used on a certain job and thus is not applicable to use in a general specification unless wide limits can be allowed or specific materials designated for use.

The data and tests reported in Mr. Burmister's paper deal only with concrete made with a gravel aggregate. It would be interesting to have data on aggregates of angular nature and on concretes with varying maximum size of aggregate and to have observations made as to whether the same flow and plasticity coefficient with the different concretes indicate similar characteristics or establish the same limits of workability measured by its suitability for satisfactory placing under a given condition.

Workability of concrete is dependent on the lubrication of the aggregate particles; the more effective the lubrication or elimination of internal friction upon deformation of the concrete mass, the more workable it will be. Water alone is not an effective lubricant unless the particles lubricated have very smooth surfaces. Water mixed with relatively insoluble powders ground to colloidal size form colloids which have varying viscosity dependent on the dilution. The proper viscosity for best lubrication of aggregate particles depends on the roughness of the particle and a more viscous paste will be necessary the rougher the particles so that it will be held in the depressions and smooth out the surface and provide a film of lubricant. The cement with the more colloidal size particles will be more efficient as a lubricant and will be more effective in utilizing the water to form lubricating colloids and less dilution of the colloids with a given water-cement ratio.

To my mind the problem of workability of concrete resolves itself into the development of a relatively simple means of measuring the internal friction within a mass of fresh concrete upon deformation.

The rate of deformation will probably be an important factor. The writer has experimented to a certain extent with apparatus based on this idea and has obtained some results which seem promising but no definite data have been obtained. Experiments are in an investigational stage in attempts to develop a suitable apparatus for testing.

MR. CLOYD M. CHAPMAN.[1]—I should like to ask how many varieties of aggregates were used, and whether any harsh, coarse aggregates or very fine aggregates were used.

MR. D. M. BURMISTER.[2]—A few tests were made on cinders which is a pretty harsh aggregate to work with. A flow of 3 or 4 in. was obtained before the mix showed any appreciable slump. Because cinder concrete is usually placed very dry and tamped in place, a better control of the mix can be obtained by means of the flow trough.

Considerable increase in workability as indicated by both the flow and the plasticity coefficient was obtained in a few tests made upon gravel concrete in which the maximum size of the coarse aggregate was reduced to $\frac{3}{8}$ in. for small test columns.

The effect of the grading of the fine aggregate on the flow and the plasticity coefficient for any given water content is brought out quite well in Figs. 3, 5 and 6 of the paper, although the difference in the grading of the sand was not very large. Figure 4 indicates the same thing in a slightly different way.

MR. J. C. PEARSON.[3]—I have had no opportunity to experiment with this apparatus, but certainly in comparison with other types of apparatus we have used, it is to be commended for its simplicity. I should like to ask Mr. Burmister whether his experience shows that the device is sensitive enough to indicate differences in workability due to cements alone. In other words, if one tested concretes with cement as the only variable, would his device show differences in workability?

MR. BURMISTER.—I cannot say very definitely as to that. I have noticed, however, an increase in the plasticity coefficient for certain cements over the past two years, along with an increase in compressive strength, which seemed to result from a finer grinding of the cement. The high-early-strength cements show similar results. By way of comparison, the effect of the addition of a small amount of Celite is brought out clearly in Fig. 7. It is probable that in a series of tests cements would show measurable differences as a result of the grinding and to some extent of their chemical constituents.

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[1] Consulting Engineer, New York City.

[2] Instructor in Civil Engineering, Columbia University, New York City.

[3] Director of Research, Lehigh Portland Cement Co., Allentown, Pa.

MR. F. R. WICKS.[1]—I should like to ask Mr. Burmister if he has done some work with crystalline talc.

MR. BURMISTER.—It was too brief a test to be very conclusive.

MR. WICKS.—It was my recollection that certain tests carried out at Columbia University gave data that indicate that the Burmister apparatus afforded a little more consistent results than the penetrometer and the slump tests. We find that in work on concrete, both with and without admixtures, it is possible to compare the slumps and the workability of two kinds of plain concrete or two kinds of concrete containing the same admixture, and that the slump then is a measure of workability; but if you try to compare one admixture with another or one admixture with a plain concrete as to slump, or as to flow table results or penetrometer tests, they are not accurate and do not give an indication of the action of the concrete under construction conditions.

For that reason, in our work we more or less discarded the penetrometer and flow table entirely and have recognized the fact that we would get a little less slump even though we have a little more workability in actual construction work. Hence this apparatus is indeed welcomed by many of us because it has huge possibilities of working out things which we are not able to work out now.

MR. BURMISTER.—Curves similar to those in Fig. 7 were obtained from two series of tests using plain 1:2:4 concrete in comparison with 1:2:4 concrete with an admixture of 3 per cent crystalline talc. These curves showed that a little more water was required to produce the same flow when talc was used and also that there was an increase in the plasticity coefficient as a measure of workability.

MR. G. A. SMITH.[2]—We have for several years recognized and considered the question of segregation as being an inherent characteristic of a concrete mix, and as being directly related to the question of workability. Mr. Burmister has indicated that his flow trough has pointed out the tendencies for a concrete mixture to segregate. I am of the opinion that a slight modification in Mr. Burmister's apparatus might be worthy of consideration in studying the question of segregation.

Instead of having the gate entirely removable, have it arranged so that it may be closed during the filling and then opened a small amount so that during the jolting operation the mortar may be free to flow out. By restraining movement of the coarse aggregate and permitting a removal of the mortar matrix, I believe there is a good possibility of being able to obtain some idea of the relative segregating tendencies of concrete mixtures.

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[1] Consulting Engineer, Pacific Coast Talc Co., Los Angeles, Calif.

[2] Hydraulic Cement Engineer, Research and Development Dept., Johns-Manville Corp., Manville, N. J.

MR. BURMISTER (*author's closure by letter*).--The suggested method of loading the trough is a departure from the usual standardized practice, in which all kinds of concrete receive the same treatment. It was adopted as an approach to field practice in which the concrete is worked as little as possible and yet produces a reasonably homogeneous mass. As a result the concrete is tested under the most favorable conditions. If it is desired to determine the effect of more working upon the properties and behavior of any given concrete, it should be made as a separate test. By so doing, more valuable information is obtained and a direct comparison can be made of the effect of such handling and working upon the concrete. Such a test can be made upon a sample of the same concrete by giving 15 back-and-forth rocking cycles to the trough before pulling the gate and making the drop tests. A comparison of the flows and general behavior of the concrete in the two tests brings out the segregation and compacting resulting from such treatment. Two slats of wood clipped to the side rails of the trough will prevent the concrete from slopping over while making the rocking test.

By using the rocking method of compacting the concrete in the regular test, each kind of concrete is given quite a definite and consistent amount of handling and working, as indicated by the concrete flowing together, showing a wet surface, and quaking. More working than is necessary to do this causes excessive compacting and detracts from the value of the test. The limits of workability are quite definitely defined in the test. A concrete which does not show some quaking could not be satisfactorily placed except by tamping or by mechanical vibratory methods. And a concrete which required no rocking would be too wet and sloppy to produce a uniform product. The concrete mass can be dilated by the addition of more mixing water to obtain greater flowability only to such a point as it is able to retain all of this water. When the breakdown in capillarity or surface tension occurs as indicated by leaking of water from the mass, a uniform homogeneous product cannot be obtained and the undesirable qualities of the concrete become accentuated.

Each test that is made upon the concrete helps to define its properties more completely. The figures and indications obtained from the tests reflect the properties and behavior of the concrete and are useful for the design and comparison of concrete mixtures.

# TEMPERATURES DEVELOPED IN MASS CONCRETE AND THEIR EFFECT UPON THE COMPRESSIVE STRENGTH

By R. E. Davis[1] and G. E. Troxell[2]

## Synopsis

This paper, first, briefly summarizes the results of field observations made by various investigators upon mass concrete structures, such as dams, bridge piers and the like, to determine the temperatures developed during the hardening period, and, second, describes laboratory tests made under the direction of the authors to determine compressive strengths of concrete cured under conditions duplicating those to be found within the mass during the hardening period, as well as observations on insulated specimens for the purpose of obtaining the time-temperature relations existing during the early ages of the hydration process.

In this latter portion of the paper are given the results of three series of tests made in the Materials Testing Laboratory of the University of California under conditions, both as regards moisture and as regards temperature, simulating those which exist in mass concrete during the period of heat evolution, these tests being preliminary to a more extensive program now in progress. In the study of compressive strengths a comparison with normal concretes is secured through tests made upon specimens cured at normal temperature.

The object of these three preliminary series of tests has been to provide a foundation for more exhaustive investigation, looking to the determination of the various physical properties of concrete cured in large masses. It has been sought to determine (1) the relation between setting temperature and time for concretes of varying cement ratio, (2) the effect of mass curing conditions upon the compressive strength, through comparison with similar concretes cured at normal temperature, (3) the effect of brand of cement upon the time-temperature relation and upon the compressive strengths of concrete subject to mass curing conditions, and (4) the effect of the initial temperature of concrete upon its strength when cured under mass conditions.

The paper shows (1) that heat in considerable quantities is generated during the hydration of all normal portland cements, (2) that even for lean mixes this heat of chemical reaction is sufficient to increase substantially the temperature of concrete in large masses and thick sections over the temperature of the mix at the time of pouring, (3) that this temperature rise is proportional to the quantity of cement present in the mix, (4) that the magnitude of the temperature rise as well as the shape of the time-temperature curve is influenced by the kind or quality of the cement, (5) that the compressive strength of concrete of small dimensions cured at normal temperature is no criterion to the compressive strength of an identical concrete cured in a large mass where

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[2] **Associate Professor of Civil Engineering, University of California, Berkeley, Calif.**

the chemical heat of hydration produces a marked increase in temperature, (6) that under normal conditions mass-cured concrete at early ages is substantially greater in strength than normal cured concrete but that at the later ages the reverse is true, (7) that the initial temperature of mass concrete has a marked effect upon the time-strength relation of mass concrete, and (8) that the time-strength relation of mass concrete differs greatly with the kind or quality of the cement.

The results presented point to the need for further research in this important field—a field where reliable information is so meager at the present time.

## INTRODUCTION

The setting of portland cement is accompanied by an evolution of heat, and in any mass of concrete the hardening process is accompanied by a rise in temperature. During the early ages, when the hydration of the cement is proceeding most rapidly, the temperature rises at a corresponding rate. Later, as the process of hydration becomes retarded, there is a time when the rate of heat generation declines until it just equals the rate of heat dissipation, when the maximum temperature is reached. Thereafter, as the rate of hydration becomes increasingly less, the temperature of the mass gradually declines, approaching that of the surrounding medium.

Other things being equal, the increase in the temperature of a mass of concrete varies as the size of the mass. For the smaller structures, for which the elements are of relatively thin section, this temperature rise is but a few degrees and is of no particular significance when compared with the effects of normal variations in the temperature of the surrounding atmosphere. However, in more massive structures, such as dams, heavy foundations, thick tunnel linings, and the like, this temperature rise is large and it becomes an important factor to reckon with, not only because of the deformations and stresses which it may produce within the structure but because of the possible effect of these high temperatures during the curing period upon the properties of the concrete itself. In order to proceed intelligently with the design of these massive structures, not only do we need to know the strengths and temperatures of the concrete under conditions which will obtain within the mass, but also we need to know the elastic and plastic properties of the concrete in both tension and compression under these same conditions as well as the effect of environment upon the thermal and hygral properties.

While it has long been known that the hardening of concrete is a chemical process accompanied by the liberation of heat, and there are abundant time-temperature records for dams and other mass concrete structures, there is great lack of information concerning all those other properties which are so necessary to rational design. Here is indeed a fertile ground for research.

The authors in preparing this paper at once recognize that its scope is limited, that its findings are most incomplete, but at the same time feel that the data presented are of sufficient significance to be worthy of publication, if for no other reason than because they may serve to emphasize the necessity for additional facts. Extended experimental programs to determine the properties of mass concrete and the factors which influence these properties are being formulated, but the field is large, with many ramifications.

This paper describes tests made during the past two years in the Materials Testing Laboratory of the University of California upon concretes subjected during the hardening period to conditions closely paralleling those which would obtain within a concrete mass of large volume. The tests have been made for two purposes: first, to determine the time-temperature relation as indicated by time-temperature curves, and, second, to determine the compressive strengths of the concretes subjected to the time-temperature and moisture conditions obtaining within a mass and to compare these results with the compressive strengths of corresponding concretes cured under normal conditions. As a preliminary, there are briefly summarized some of the more important temperature data obtained by other investigators through observations of field structures, and mention is made of other significant laboratory tests where time-temperature observations have been made. It appears that there are almost no published data regarding the strength of mass-cured concrete.

Some of the factors which may influence the maximum temperature attained by concrete due to the chemical heat liberated or which may affect the time required to reach the maximum temperature include (1) chemical composition of the cement, (2) fineness of the cement, (3) richness of the mix, (4) water-cement ratio, (5) character of aggregate, (6) initial temperature of the concrete, (7) temperature and character of the surrounding medium, (8) the size and shape of the concrete mass, and (9) the speed of placement of the concrete. A variation in any one of these nine factors may influence to greater or lesser degree the maximum temperature reached by the concrete and may therefore exert more or less influence upon the strength, elasticity, durability and other properties of the mass and may also affect the intensity and distribution of stresses and strains within the mass.

Even though the use of normal portland cements and normal mixes for structures of thin section result in no appreciable rise in temperature, it is a matter of record that in massive concrete structures large temperature increases have occurred subsequent to placing of concrete, in some cases amounting to as much as 100° F., and not infrequently a month or more elapses before the concrete attains its maximum temperature.

## SOME OBSERVED TEMPERATURES

The early tests on mass concrete were concerned primarily with the maximum temperature rise resulting from the chemical activity of the cement.

Tests to show the effect of the cement upon the temperature rise were made by the Watertown Arsenal in 1901, the specimens being 12-in. neat cement cubes, using 27 different brands of portland cement. These specimens, having an average initial temperature of 77° F., were cast in wooden forms with their tops exposed to the air. The maximum temperature developed by any of the cubes was 218° F., showing a rise of 141° F. in 13 hours. The lowest maximum temperature for any of the cements was 117° F., giving a rise of 40° F.

The earliest tests of record upon a massive structure are those reported by Merriman on the Boonton Dam in New Jersey in 1903. The interior of this dam was of cyclopean concrete composed of about 50 per cent of large stones and 50 per cent of a 1:3:6 mix of concrete, so that the cement content was probably only 2 sacks per cubic yard. The concrete in this structure was placed in relatively thin layers so that even toward the top where the thickness was less than 20 ft., the increase in height was at the rate of less than 1 ft. per day. These placement conditions permitted rapid dissipation of the chemical heat, but even so, a maximum rise of 19° F. was observed 13 days after placing the electric thermometers in position.

More recent observations of temperature developed in mass structures, employing the richer mixes now in common use, show much higher maximum temperatures than those observed previously. It is reported that in a section of tunnel lining for the Hetch Hetchy Water Supply of San Francisco, for which the mix contained 9 sacks of cement per cubic yard, the temperature rose to 180° F., a rise of approximately 110° F. above the initial temperature. Also, during the construction of the strong room for the Bank of England, a temperature rise of 100° F. was observed.

Temperature rises of 70° F. or more were observed in (1) a slab of $1:2\frac{1}{2}:5$ concrete, 15 ft. thick, added to the downstream face of the Spaulding Dam in California during 1919; (2) the Cleveland, Cincinnati, Chicago and St. Louis Railway Co. bridge pier at Sidney, Ohio, in 1923; (3) the Emigrant Creek Dam in Oregon, constructed in 1924, which has a base thickness of 19.5 ft. and was made using a 1:2½:5 mix containing 1.25 bbl. of cement per cubic yard of concrete; (4) the anchorages for the new Hudson River bridge in New York City, which were placed in 1928; (5) a shaft constructed by the Board of Water Supply of New York City; (6) the Calderwood Dam built of a 1:3:4½ mix, containing about 1.1 bbl. of cement per cubic yard; and (7) the Wilson Dam on the Tennessee River which was built in 1923 of concrete containing 1.25 bbl. of cement per cubic yard.

In every case of these massive constructions for which there are temperature records, it was observed that the maximum temperature occurred several days after the placing of the concrete; in some cases the period of rise in temperature lasted for more than one month.

Many other records of more recent date show that rises of temperature of 50° F. due to the hydration of the cement are quite common. Structures which may be included in this group are (1) the Hollingsworth and Whitney Co.'s thick canal walls, over 26 ft. high, at Madison, Maine, a construction using a 1:3:5 mix; (2) the spillway arch bridge at the Ashokan reservoir built by the Board of Water Supply of New York City, the thickness at the abutment being 6 ft. 3 in.; (3) the 160-ft. span arch bridge near Eatonton in Putnam County, Georgia, made with a 1:2:4 mix; (4) Kensico dam, a gravity structure 307 ft. high, built at Valhalla, N. Y. in 1913, using a cyclopean masonry, 27 per cent of the mass being of large stone and the remainder being of a 1:3:6 mix of concrete; (5) the auxiliary structures in connection with the Chute a Caron dam in Quebec, Canada, for which the mix was 1:2.3:4.8, the maximum size of aggregate being 6 in.; (6) the Gibson dam in Montana, which is 54.5 ft. thick at the base, built in 1928 of concrete containing 1 bbl. of cement per cubic yard, with maximum temperature occurring at the age of 13 days; (7) the La Jogne dam in Switzerland, which was built in 1919 and 1920, using a 1:3.15:5.05 mix, the dam being 31 ft. thick and the maximum temperature occurring after 6 days; and (8) the Owyhee dam in Oregon, now under construction by the U. S. Bureau of Reclamation, using a 1:2.7:6.45 mix by weight. In this latter structure, which is 405 ft. high and 244 ft. thick at the base, the maximum temperature was developed after 3 weeks.

Tests by Bates at the U. S. Bureau of Standards and by Merriman of the Board of Water Supply of New York City, upon small samples of 32 brands of neat cement, have shown that, under the conditions of the tests where only partial insulation was secured, there is a marked difference among cements in the matter of temperature-rise during the first day or so after the beginning of hydration.

Whether tests upon larger and more perfectly insulated samples would show the same marked difference between cements is problematical.

The temperature records just cited show conclusively that all mass concrete during the curing period is subjected to temperature conditions markedly different from normal atmospheric temperatures. Since the rate of hydration of cement is known to be influenced by the temperature and since strength and other properties of concrete depend upon the degree of hydration, it immediately becomes evident that to determine these properties with any degree of certainty, conditions obtaining within the mass must be duplicated, both as regards temperature and as regards moisture.

## OUTLINE OF CALIFORNIA TESTS

The University of California tests were begun in 1929 and are still in progress. As this is written, more than two thousand specimens have been tested, but these are regarded as merely preliminary to a much more comprehensive program which it is expected will extend over several years. So far, the tests have been concerned with time-temperature records and with determinations of compressive strength. Temperature observations have extended over periods up to two months and compressive strengths have been determined at various ages from 1 day to 1 year.

The tests here reported are in three series:

For series 1, the specimens were insulated 3-ft. cubes in each of which there were embedded twelve 6 by 12-in. cylinders in sealed containers. Through the use of thermocouples, scattered throughout the mass of each cube, continuous records of temperature were obtained. At various ages the insulation was removed, the cubes were disrupted and the embedded 6 by 12-in. cylinders were tested in compression. These tests were supplemented by parallel tests made upon 6 by 12-in. concrete cylinders in sealed containers, cured under temperature conditions identical with those developed in corresponding 3-ft. cubes. For this portion of the tests the specimens were stored in insulated chests where the temperature could be regulated as desired. In this manner the time-temperature cycle of the mass was duplicated in small specimens, and at various ages these small specimens were tested in compression, as were those taken from the 3-ft. cubes.

The purpose of this series was to determine the time-temperature relations for mass concrete, the effect of richness of mix upon the setting temperature, and the compressive strengths at various ages of concrete which has been subjected to conditions, both as regards temperature and moisture, equivalent to those which will be found within a mass of considerable extent.

For series 2, tests were made to determine the effect of brand of cement upon concrete-setting temperatures and upon related compressive strengths. The investigation included nine California portland cements, purchased in the open market. For all of these cements the proportions of concrete materials and the water-cement ratio were maintained constant, hence the only variable was among the cements themselves.

To determine the time-temperature relation for these nine cements, 18-in. insulated cubes were cast at approximately 70° F. and were maintained in a constant-temperature room, where by means of embedded thermocouples temperature observations of the concrete were made at frequent intervals. To determine the effect of curing temperatures upon the compressive strength, concrete cylinders using these nine cements were cast in sealed cans, some of which were continuously maintained at 70° F. during the hardening period and others of which were subjected to a time-temperature cycle calculated to be the equivalent of that which would be experienced in a large mass.

TABLE I.—CONCRETE MIXES, SERIES 1.

| Mix | Cement, sacks per cu. yd. | Water, gal. per sack of Cement | Water-Cement Ratio | Approximate Mix by Volume | Proportions by Volume | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | Santa Cruz Cement | Sand | $\frac{1}{4}$ to $\frac{3}{4}$-in. Gravel | $\frac{3}{4}$ to $1\frac{1}{4}$-in. Gravel |
| No. 1 | 9.46 | 4.5 | 0.60 | 1:3 | 1 | 1.20 | 0.90 | 0.92 |
| No. 2 | 6.15 | 6 | 0.80 | 1:5 | 1 | 2.11 | 1.54 | 1.57 |
| No. 3 | 4.78 | 8 | 1.07 | 1:7 | 1 | 2.82 | 2.07 | 2.08 |

For series 3, tests were made to determine the effect of initial temperature upon the compressive strength of concrete. The specimens were cylinders in sealed cans. Three mixes were employed at three initial temperatures. For each mix and initial temperature, half of the specimens were cured at 70° F. and the remainder were cured under temperature conditions calculated to simulate those which would exist in a large mass poured at the given initial temperature. At various ages, the specimens were tested in compression.

## EFFECT OF RICHNESS OF MIX UPON TEMPERATURE AND STRENGTH—SERIES 1

For the tests of series 1, three different mixes of gravel concrete were used, the proportions being as shown in Table I. The sand was well graded from 0 to $\frac{1}{4}$ in. and had a fineness modulus of 3.03. The $\frac{1}{4}$ to $\frac{3}{4}$-in. and the $\frac{3}{4}$ to $1\frac{1}{4}$-in. gravels had fineness moduli of 6.33 and

7.81, respectively. For each mix the consistency was such as to produce a 4-in. slump.

For each of the mixes, 3-ft. cubes were cast in wooden forms of 2-in. lumber placed inside of a second form of 1-in. lumber, the space between being filled with dry sawdust, 6 in. thick for the 1:3 mix and 12 in. thick for the 1:5 and 1:7 mixes; also, the top of each cube was similarly insulated with sawdust.

As each cube was poured, twelve 6 by 12-in. cylinders of the same mix, in sealed containers, were embedded in the cube, as close to the center as possible—four in an inner circle and eight in an outer circle. Iron-constantin thermocouples were embedded in several of the cylinders and at numerous other points within the cubes in order to determine the maximum temperatures developed within the cubes and also to ascertain temperature gradients within the mass. For each cube, frequent observations were made of the temperatures within the mass and of the surrounding air.

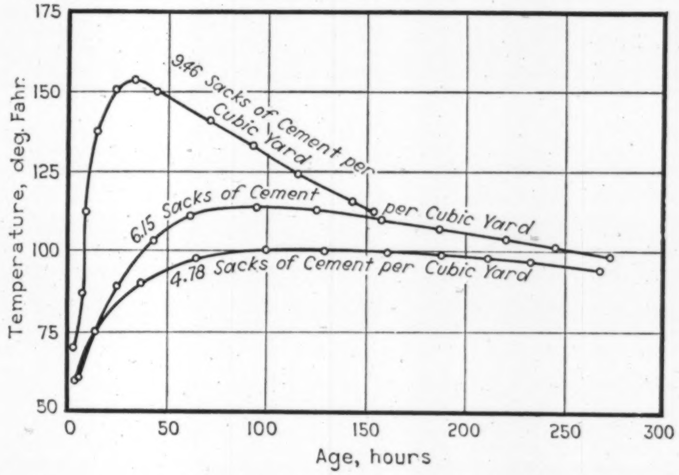

FIG. 1.—Temperatures Developed at Centers of 3-ft. Insulated Cubes, Series 1.

Control specimens consisted of 6 by 12-in. cylinders similar to and cast at the same time as those embedded in each cube. For the 1:3 mix, some of the control cylinders, after the first 24 hours, were stored without forms in a fog curing room at 70° F.; the remaining control cylinders were stored under normal atmospheric conditions in sealed cans. For the 1:5 and 1:7 mixes, some of the control cylinders were similarly removed from their molds at 24 hours and were then stored in a fog room at 70° F., but the remaining cylinders were stored in sealed containers at 70° F.

The time-temperature curves for the centers of cubes of the three concrete mixes are shown in Fig. 1. It will be observed from the figure that, regardless of the mix, the temperature rise was rapid during the first day. It is instructive to note that during this period the richest mix increased in temperature at the rate of about 10° F. per hour, while the lean mix increased in temperature at the rate of about 3° F. per hour. The largest maximum temperature rise, 83° F., was developed by the richest mix at the age of 33 hours; the smallest maximum temperature rise, 42° F., was developed by the lean mix after 97 hours.

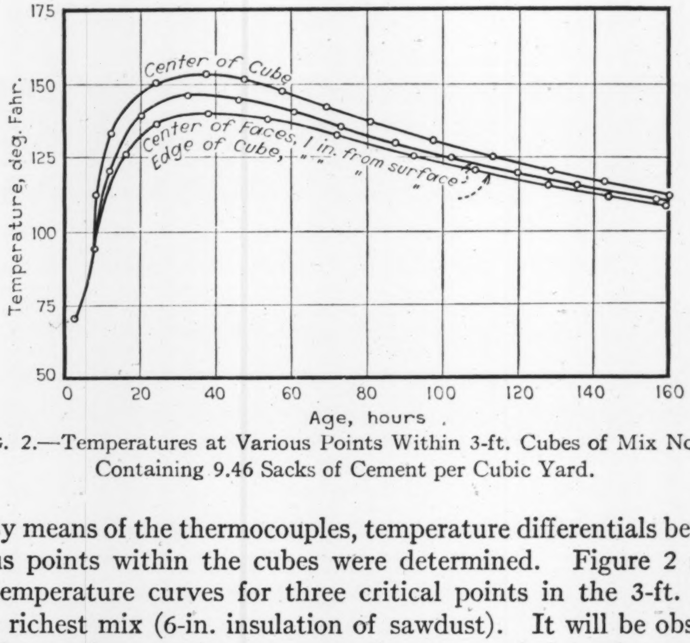

FIG. 2.—Temperatures at Various Points Within 3-ft. Cubes of Mix No. 1 Containing 9.46 Sacks of Cement per Cubic Yard.

By means of the thermocouples, temperature differentials between various points within the cubes were determined. Figure 2 shows time-temperature curves for three critical points in the 3-ft. cubes of the richest mix (6-in. insulation of sawdust). It will be observed that the maximum temperature at the center of the cube exceeded that developed 1 in. from the surface at the edges of the cube by 13° F. For the leaner mixes, with 12 in. of sawdust (Nos. 2 and 3, Table I), the temperature drop from the center of the cube to the points near the edges was only 2° F.

Although moderately high temperatures were observed in the 3-ft. cubes, it is apparent from a study of Fig. 2 that the sawdust insulation of the cubes was but imperfectly effective in preventing heat losses and that higher temperatures would occur in massive structures of the same mixes provided the concrete were placed with the speed usual for such structures and under corresponding initial temperature conditions.

The 3-ft. cubes were disrupted at ages varying from 1 day to 1 year, the embedded 6 by 12-in. cylinders being removed and tested in compression along with the corresponding control cylinders.

Table II shows the average compressive strengths of each group of embedded specimens and the control cylinders for the three mixes.

The most striking thing shown by Table II is the early development of strength by the specimens cured within the cube at a high temperature, in comparison with the strengths developed by similar specimens cured at 70° F. But it also appears that at the later ages, the strengths of the specimens cured at 70° F. are in most cases appreciably greater than for the cylinders cured within the large cubes. These results lead to the conclusion that mass concrete attains higher strengths at early ages than does similar concrete poured in thin sections where the chemical heat is dissipated, but that under favorable conditions as regards moisture, ultimately the concrete cured at normal temperatures will develop greater strength than the mass concrete. Any early drying out of the concrete placed in thin sections would, of course, interrupt the further development of strength, with

TABLE II.—TEMPERATURES OF 3-FT. CUBES AND STRENGTHS OF CYLINDERS, SERIES 1.

| Mix | Approximate Mix | Proportions by Volume | Water, gal. per sack of Cement | Cylinders | Storage | Room Temperature, deg. Fahr. | Cube Temperature, deg. Fahr. |  |  | Hours to Maximum Temperature | Compressive Strength, lb. per sq. in. |  |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  |  |  |  |  | Initial | Maximum | Rise |  | 1-Day | 3-Day | 7-Day | 28-Day | 90-Day | 1-Year |
| No. 1 | 1:3 | 1:1.20:1.82 | 4.5 | Inner | Cube | 77 | 71 | 154 | 83 | 33 | 3110 | 3130 | 3720 | .... | 4120[b] | 4030[b] |
|  |  |  |  | Outer | Cube | 77 | 71 | 153 | 82 | 33 | 3080 | 3270 | 3710 | .... | 3960[b] | 3950[b] |
|  |  |  |  | Control | Laboratory Air[a] | .. | .. | .. | .. | .. | 995 | 2530 | 3200 | .... | 5010[b] | .... |
|  |  |  |  | Control | Fog at 70° F. | .. | .. | .. | .. | .. | 1375 | 3000 | 3870 | .... | 5750[b] | 5220[b] |
|  |  |  |  | Control | Air at 70° F.[e] | .. | .. | .. | .. | .. | .... | .... | .... | .... | .... | 4480 |
| No. 2 | 1:5 | 1:2.11:3.11 | 6 | Inner | Cube | 68 | 60 | 115 | 55 | 94 | .... | .... | 3340 | 3840 | 3795 | 4030 |
|  |  |  |  | Outer | Cube | 68 | 60 | 115 | 55 | 94 | .... | .... | 3260 | 3780 | 3770 | 3950 |
|  |  |  |  | Control | Fog[c] | .. | .. | .. | .. | .. | .... | .... | 2120 | 3620 | 4170 | 5220 |
|  |  |  |  | Control | Fog at 70° F.[d] | .. | .. | .. | .. | .. | .... | .... | 2610 | 4230 | 4570 | 4480 |
| No. 3 | 1:7 | 1:2.82:4.15 | 8 | Inner | Cube | 67 | 59 | 101 | 42 | 97 | .... | .... | .... | 2890 | 3240 | 3280 |
|  |  |  |  | Outer | Cube | 67 | 59 | 101 | 42 | 97 | .... | .... | .... | 2890 | 3160 | 3360 |
|  |  |  |  | Control | Fog[c] | .. | .. | .. | .. | .. | .... | .... | .... | 2620 | 3420 | .... |
|  |  |  |  | Control | Fog at 70° F.[d] | .. | .. | .. | .. | .. | .... | .... | .... | 3180 | 3850 | 3830 |
|  |  |  |  | Control | Air at 70° F.[e] | .. | .. | .. | .. | .. | .... | .... | .... | .... | .... | 4050 |

[a] Specimens in air at about 60° F. stored in sealed molds.

[b] After 7 days storage as indicated, cylinders were removed, coated with asphalt and stored at 70° F. and 70 per cent relative humidity.

[c] Specimens stored in fog at 70° F. in open molds for 28 days. Cylinders then sealed and stored at 70° F. and 70 per cent relative humidity.

[d] Molds removed after 1 day.

[e] Specimens sealed and stored at 70° F. and 70 per cent relative humidity.

the consequent result that similar concrete of massive section might easily exhibit a higher strength even at the later ages.

Tests supplementing those just described were made for the two richer mixes, the specimens being 6 by 12-in. cylinders. For each of these two mixes there were three groups of specimens as follows:

The specimens of the first group, cast in sealed containers, were cured in specially constructed insulated chests equipped with heat controls and were subjected to the same temperature conditions as the concrete in the corresponding 3-ft. cubes; the specimens of the second group were stored without forms in a fog room at 70° F., to serve as standard control specimens; and the specimens of the third group were stored in containers in air at 70° F. and 50 per cent relative humidity.

TABLE III.—SETTING TEMPERATURES OF 18-IN. CUBES USING NINE DIFFERENT CEMENTS, SERIES 2.

| Group | Brand of Cement | Temperatures, deg. Fahr. | | | Age at Maximum Temperature, hours |
|---|---|---|---|---|---|
| | | Initial | Maximum | Rise | |
| A | 1 | 68 | 125 | 57 | 33 |
| | 2 | 65 | 127 | 62 | 32 |
| | 3 | 69 | 121 | 52 | 28 |
| | 4 | 68 | 119 | 53 | 40 |
| B | 5 | 64 | 128 | 64 | 25 |
| | 6 | 69 | 130 | 61 | 28 |
| | 7 | 66.5 | 132 | 65.5 | 32 |
| C | 8 | 69 | 133 | 64 | 30 |
| | 9 | 68.5 | 139 | 70.5 | 29 |

The specimens of these three groups were tested at ages varying from 1 day to 1 year. For the richest mix, the specimens cured at high temperatures gained strength quite rapidly for 3 days but changed very little from then on. A slight retrogression between 7 and 28 days was fully recovered at the end of a year. The specimens cured in fog at 70° F. and those sealed in cans at 70° F. at the early ages gained strength at a slower rate, but between the ages of 7 and 28 days they became stronger than those cured at the higher temperatures, and the advantage in favor of the specimens cured at normal temperatures became more pronounced with time, being 15 to 30 per cent greater at the later ages.

In general, the results of these tests agreed closely with the results of corresponding tests upon cylinders embedded in the 3-ft. cubes.

### INFLUENCE OF CHARACTER OF CEMENT—SERIES 2

The tests of series 2 were made to determine the influence of character of the cement upon the maximum temperature developed under conditions simulating those which exist in mass concrete, and also to ascertain the effect of this temperature upon the compressive strength. Nine brands of cement were tested.

The concrete mix was 1:1.50:1.85, the proportions being by weight. The cement content of the mix was 8.8 sacks of cement per cubic yard. The fine aggregate was a well-graded coarse sand, ranging from 0 to $\frac{3}{8}$ in., having a fineness modulus of 3.71, and the coarse aggregate was a crushed stone, graded from No. 14 to $\frac{1}{2}$ in., and having a fineness modulus of 5.82. The water-cement ratio was kept constant at 0.67, producing a slump of from 3 to $3\frac{1}{2}$ in.

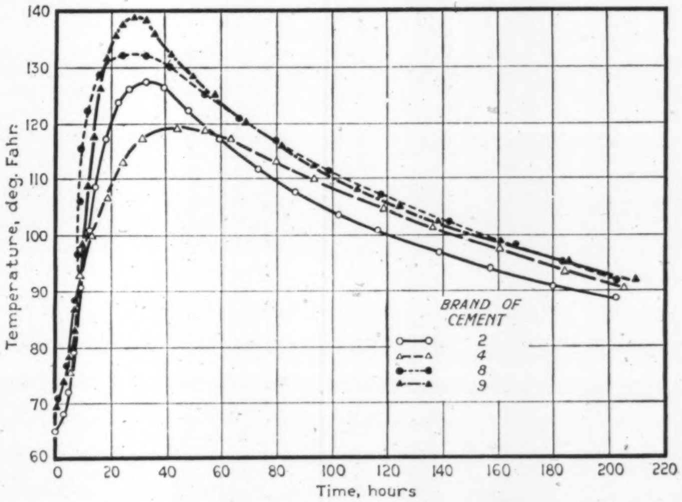

FIG. 3.—Temperatures Developed at Centers of 18-in. Insulated Cubes Containing 8.8 Sacks of Different Brands of Portland Cement per Cubic Yard.

The specimens were 18-in. cubes, insulated on all sides with 12 in. of dry sawdust, all housed in a room where the air was kept constant at 70° F. during the progress of the tests. Thermocouples embedded at the center of each cube were used to determine the setting temperatures at frequent intervals. The critical data of the tests are shown in Table III.

The time-temperature curves for four of the nine mixes cast in the 18-in. insulated cubes are given in Fig. 3. The curves which have been chosen represent the extremes among the cements either as to maximum temperature attained or as to characteristics of the time-temperature cycle. Thus the curves for the remaining 5 cements would lie intermediate to those shown.

From a study of Fig. 3 and Table III, it will be observed that among the nine cements there was a variation of 20° F. in the maximum setting temperatures, the highest maximum temperature being 139° F. and the lowest 119° F. The time required to develop the maximum temperatures after mixing ranged from a maximum of 40 hours to a minimum of 25 hours.

For each of the nine cements, the effect of curing conditions upon the compressive strengths of concretes was determined for each of the following two conditions of storage: (1) air at 70° F. and 50 per cent relative humidity, and (2) high-temperature chest in which the temperature was varied according to a time-temperature curve conforming in shape with that of the corresponding 18-in. insulated cube but with a maximum temperature equal to that which would have been reached in a large mass. These maximum temperatures varied from 163 to 180° F., depending upon the brand of cement.

TABLE IV.—COMPRESSIVE STRENGTHS (LB. PER SQ. IN.) AT VARIOUS AGES FOR NORMAL TEMPERATURE AND HIGH TEMPERATURE CURE SIMULATING MASS CONCRETE CONDITIONS, SERIES 2.

| Group | Brand of Cement | Conditions of Storage | | | | | |
|---|---|---|---|---|---|---|---|
| | | Air at 70° F. | | | High Temperature | | |
| | | Age 1 Day | Age 3 Days | Age 7 Days | Age 1 Day | Age 3 Days | Age 7 Days |
| A | 1 | 545 | 1355 | 2400 | 1050 | 2060 | 3080 |
| | 2 | 580 | 1925 | 2930 | 1755 | 2340 | 2645 |
| | 3 | 1190 | 3220 | 3930 | 3065 | 4090 | 4670 |
| | 4 | 530 | 1910 | 2990 | 1700 | 2380 | 2610 |
| B | 5 | 1060 | 1960 | 2985 | 2890 | 3485 | 3755 |
| | 6 | 865 | 2135 | 3290 | 2375 | 2765 | 3380 |
| | 7 | 845 | 2455 | 3550 | 2395 | 3020 | 3170 |
| C | 8 | 1310 | 2890 | 4020 | 2715 | 4100 | 4220 |
| | 9 | 1230 | 2900 | 3940 | 2640 | 3425 | 3450 |

For all nine of the cements, the compression specimens were 3 by 6-in. cylinders cast in sealed cans. For each of the conditions of storage, specimens were tested at the ages of 1, 2 and 7 days. The results of the tests are shown in Table IV.

It is seen that at the ages of 1 and 3 days, the cylinders cured at the high temperatures exhibited higher strengths than those cured at 70° F. For five of the nine cements at the age of 7 days, the strengths of the high-temperature-cured cylinders exceeded the strengths of the cylinders stored at 70° F.

## EFFECT OF INITIAL TEMPERATURE—SERIES 3

For a given mix, the initial temperature of the concrete practically determines the maximum temperature which will be attained in massive sections, since it appears that the magnitude of temperature rise is not greatly affected by the temperature of the mass at the time of placement. Therefore, to determine the effect of the initial temperature of the mix—through its relationship to the maximum temperature attained—upon the compressive strength of the concrete, three groups of specimens were prepared, the initial temperatures of the concrete for these three groups being respectively 40, 70 and 100° F. The specimens were 3 by 6-in. cylinders.

At each of these temperatures, three concrete mixes were prepared, with water-cement ratios corresponding to those of series 1; that is, 4½, 6 and 8 gallons of water per sack of cement, and with a uniform slump of 4 in. The fine aggregate was a well-graded sand ranging from 0 to ¼ in., having a fineness modulus of 3.03, and the coarse aggregate was a well-graded gravel ranging from ¼ to ¾ in., having a fineness modulus of 6.71. The mix proportions and other pertinent data are shown in Table V.

TABLE V.—CONCRETE MIXES, SERIES 3.

| Mix | Water, gal. per sack of Cement | Cement, sacks per cu. yd. | Proportions by Weight | | | Maximum Rise in Temperature, deg. Fahr.[a] |
|---|---|---|---|---|---|---|
| | | | Santa Cruz Cement | Sand | ¼ to ¾-in. Pebbles | |
| No. 1 | 4.5 | 13.18 | 1 | 0.79 | 2.28 | 104 |
| No. 2 | 6 | 8.10 | 1 | 2.30 | 3.45 | 68 |
| No. 3 | 8 | 5.37 | 1 | 3.14 | 4.14 | 50 |

[a] Produced in high-temperature chest.

For each of the three mixes and for each of the three initial temperatures, three conditions of curing were employed. Under the first condition, the specimens were stored in fog at 70° F. until the time of test, the forms being stripped at 24 hours; under the other two curing conditions, the specimens were cast in sealed cans so as to prevent the loss or gain of moisture. One-half of the specimens in sealed cans were stored at 70° F., while the remaining cylinders were placed in the high-temperature chest and subjected to a time-temperature treatment to simulate conditions in mass concrete.

The rises in temperature for the three mixes which are shown in Table V were based upon the rises observed for the 3-ft. cubes of series 1, and were calculated to represent mass concrete conditions. The maximum temperatures were attained at the age of 14 days and were maintained constant until the age of 28 days, after which time the cylinders were gradually cooled to 70° F. They were then stored at this latter temperature until tested.

The complete program involves the tests for each condition of mix, initial temperature and type of cure at the ages of 1, 3, 7, 28 and 90 days, and 1 year. The results now available are presented in Table VI.

The effect of the storage condition upon the compressive strength of the specimens made of mix No. 2 at an initial temperature of 100° F. is shown by Fig. 4. The rapid development of strength at the early ages for the specimens cured at the high temperature is evident, as is also the more continuous and eventually greater development of strength at the later ages in the specimens cured at 70° F.

TABLE VI.—COMPRESSIVE STRENGTHS (LB. PER SQ. IN.) AS AFFECTED BY INITIAL TEMPERATURE AND CURING CONDITIONS, SERIES 3.

| Mix | Cement, sacks per cu. yd. | Initial Temperature, deg. Fahr. | Curing and Maximum Temperature Conditions | Age at Test, days | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | 1 | 3 | 7 | 28 | 90 |
| No. 1............ | 13.18 | 40 | Fog, 70° F. | 538 | 1622 | 2257 | 3922 | .... |
| | | | Sealed, 70° F. | 1072 | 2441 | 3397 | 3976 | .... |
| | | | Sealed, 144° F. | 1184 | 4014 | 3689 | 4052 | .... |
| No. 1............ | 13.18 | 70 | Fog, 70° F. | 780 | 2434 | 3474 | 4535 | .... |
| | | | Sealed, 70° F. | 1030 | 3239 | 3526 | 4934 | 5859 |
| | | | Sealed, 174° F. | 2868 | 4285 | 3820 | 4452 | 4742 |
| No. 1............ | 13.18 | 100 | Fog, 70° F. | 834 | 2455 | 3455 | 4440 | 4925 |
| | | | Sealed, 70° F. | 1496 | 2758 | 3495 | 4520 | .... |
| | | | Sealed, 204° F. | 2627 | 2965 | 3062 | 3900 | 4008 |
| No. 2............ | 8.10 | 40 | Fog, 70° F. | 306 | 1294 | 2274 | 3344 | .... |
| | | | Sealed, 70° F. | 401 | 1308 | 2178 | 3570 | .... |
| | | | Sealed, 108° F. | 250 | 1686 | 3118 | 3661 | .... |
| No. 2............ | 8.10 | 70 | Fog, 70° F. | 512 | 1559 | 2545 | 3601 | 4567 |
| | | | Sealed, 70° F. | 644 | 1692 | 2550 | 4026 | 4534 |
| | | | Sealed, 138° F. | 916 | 2675 | 3240 | 3925 | 4190 |
| No. 2............ | 8.10 | 100 | Fog, 70° F. | 693 | 2095 | 3130 | 4180 | 4866 |
| | | | Sealed, 70° F. | 822 | 2045 | 2943 | 3970 | 4544 |
| | | | Sealed, 168° F. | 1610 | 2515 | 2975 | 3075 | 3380 |
| No. 3............ | 5.37 | 40 | Fog, 70° F. | 175 | 630 | 1196 | 1738 | .... |
| | | | Sealed, 70° F. | 188 | 660 | 1144 | 2200 | .... |
| | | | Sealed, 90° F. | 106 | 773 | 1625 | 2156 | .... |
| No. 3............ | 5.37 | 70 | Fog, 70° F. | 165 | 639 | 1130 | 2315 | 2525 |
| | | | Sealed, 70° F. | 236 | 665 | 990 | 1985 | 1925 |
| | | | Sealed, 120° F. | 293 | 1165 | 1590 | 1890 | 1872 |
| No. 3............ | 5.37 | 100 | Fog, 70° F. | 116 | 546 | 1261 | 2391 | 2641 |
| | | | Sealed, 70° F. | 222 | 844 | 1520 | 2609 | 2833 |
| | | | Sealed, 150° F. | 542 | 1253 | 1585 | 2137 | 2135 |

Figure 5 shows similar curves for the specimens of this same mix (No. 2) which were cast at 70° F. It will be noted that those cured at 70° F. in fog or in sealed cans did not show superior strength to those cured at high temperatures until the age of 28 days. At ages under 28 days, the excess in strength of the specimens cured at high

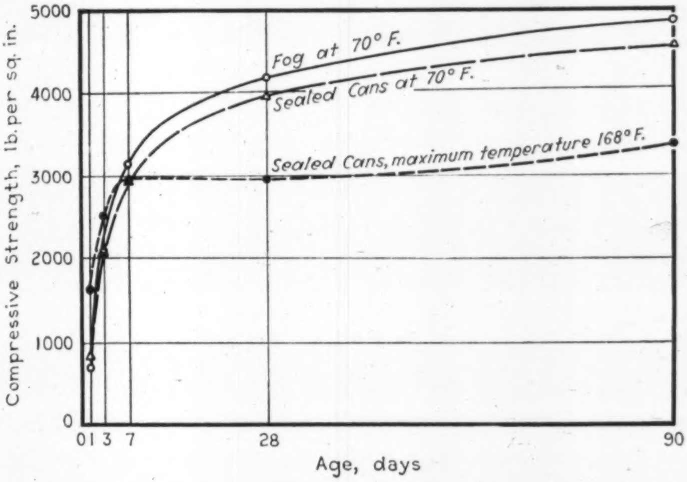

FIG. 4.—Effect of Curing Condition on the Strength of Concrete Placed at 100° F. for Mix No. 2; 8.10 Sacks Cement per Cubic Yard.

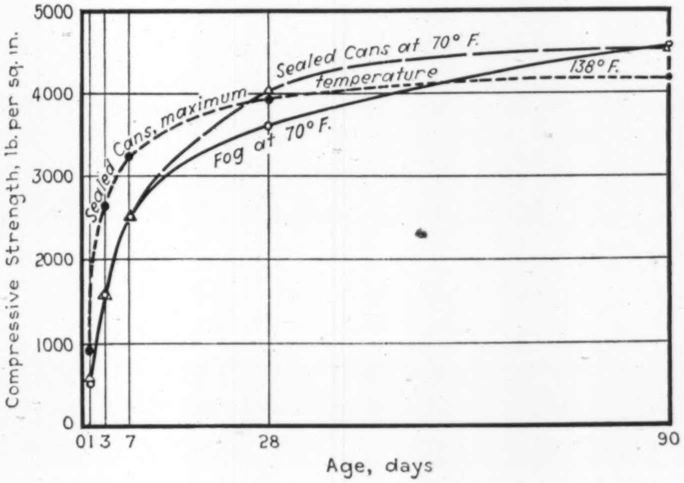

FIG. 5.—Effect of Curing Condition on the Strength of Concrete Mix No. 2 Placed at 70° F. and Containing 8.10 Sacks of Cement per Cubic Yard.

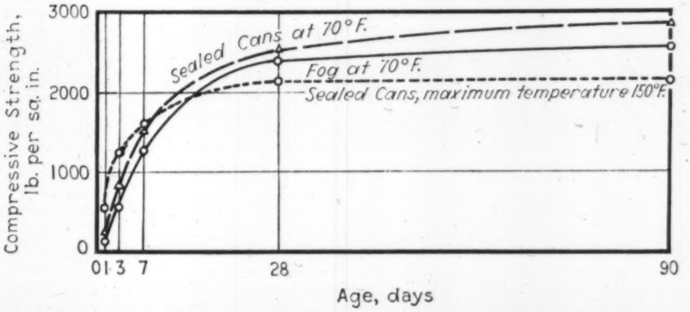

FIG. 6.—Effect of Curing Condition on the Strength of Concrete Mix No. 3 Placed at 100° F. and Containing 5.37 Sacks of Cement per Cubic Yard.

temperatures over those cured at 70° F. is even more marked for the specimens cast at 70° F. than for those cast at 100° F.

For mix No. 2 cast at 40° F., Table VI shows that up to the age of 28 days, the specimens cured at high temperatures develop greater strength than those cured in fog or in cans at 70° F.

The effect of curing conditions upon the compressive strength of concrete of the lean mix (No. 3) is shown by Fig. 6. It will be noted that the specimens cured at the high temperatures exhibit higher strengths at the early ages than do those cured at 70° F., but that the strengths are somewhat lower at the later ages. The data of Table VI show, further, that the strength-age relationship for the three conditions of storage and for the three initial temperatures of the concrete of the richer and leaner mixes Nos. 1 and 3 are not unlike those for mix No. 2, as shown in Figs. 4 and 5.

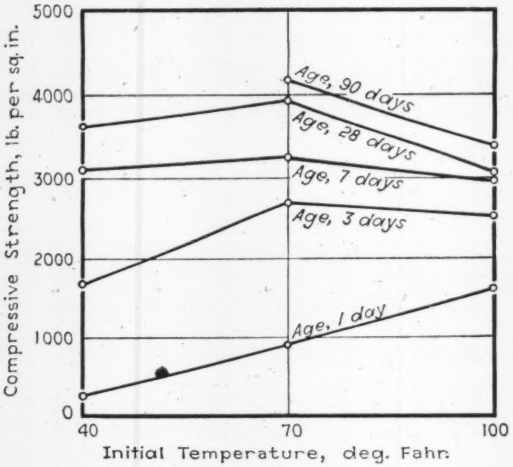

FIG. 7.—Effect of Initial Temperature on the Compressive Strength of Concrete Cured in Sealed Cans in High Temperature Chest; Mix No. 2; 8.10 Sacks Cement per Cubic Yard; Maximum Temperature rise 68° F.

Figure 7 shows the effective of the initial temperature of the batch upon the compressive strength of mix No. 2 at various ages, when cured in the high-temperature chest, the temperature rise being 68° F. For the ages of 1 and 3 days it will be observed that the higher initial temperatures with their resultant higher curing temperatures substantially increased the compressive strengths. At the later ages, however, the strengths are greater for specimens mixed at 70° F. that for those cast at 40° F. and 100° F.

For the richest mix (mix No. 1) at the early as well as at the later ages, an initial temperature of 70° F. was effective in producing higher strengths than initial temperatures of either 40 or 100° F. But for the leaner mix (mix No. 3), an initial temperature of 100° F. was equally as effective in producing high strengths at all ages as were initial temperatures of 40 and 70° F.

## CONCLUSIONS

1. Heat in considerable quantities is generated by all normal portland cements during the hydration process.

2. This heat of chemical reaction is sufficient materially to raise the temperature of concrete in large masses or thick sections above the temperature of the mix at the time of pouring. For a given concrete the magnitude of this temperature rise accompanying the hardening process and also the time interval before the maximum temperature is reached increase as the size of the mass and as the rate of placement.

3. Other things being equal, the magnitude of the temperature rise is proportional to the richness of the mix. Under normal conditions for lean mixes, such as used in dam construction temperature rises of 40 to 60° F. may be expected, and for rich mixes temperature rises of 80 to 100° F. or more have been observed in structures of thick section.

4. Other things being equal, the magnitude of the temperature rise as well as the shape of the time-temperature curve is substantially influenced by the kind or quality of the cement.

5. The time-temperature relation exhibited by a given cement in concrete of small mass, even though partially insulated against heat loss, is no criterion to the time-temperature relation which will be exhibited by the same concrete in large mass. Hence in comparing cements as to their relative heating qualities, it is essential that conditions which will obtain within the structural mass be closely simulated. Of two cements, that which shows the lesser temperature rise in a small specimen may show the greater temperature rise in the large mass, the rate of heat generation as well as the total heat generated having an important influence upon temperature change.

6. For normal mixes and normal portland cements, the maximum temperature may be reached in small specimens within less than 24 hours after casting. But in large masses, the heat of delayed hydration may be generated sufficiently slowly to increase the temperature for several weeks, and in comparing cements the rise in temperature during the early stages of the hardening process is no safe criterion to the temperature which may ultimately be reached.

7. The compressive strength of a given concrete cured at normal temperatures is no indication of the compressive strength of a concrete of identical materials in a mass of proportions sufficiently large to retain the heat of hydration and to substantially increase the temperature above normal. With a normal portland cement the strength of 1-day mass-cured concrete is always greater than (not infrequently three times as great as) 1-day normal-cured concrete. But within 7 to 28 days the strength of normal-cured concrete reaches that of the mass-cured concrete, and at the later ages the strength of the normal-cured concrete is the greater.

8. Variations in compressive strength as influenced by the kind or quality of cement are greater for mass-cured than for normal-cured concretes. Furthermore, at least for the early ages, the strength ratio of two cements in normal-cured concretes is no indication of the corresponding strength ratio in mass-cured concretes. Thus at a given age brand "A" may exhibit a substantially higher strength than brand "B" in concrete cured at 70° F., as say in a paving slab, but the reverse may be true in the concrete of a dam or other structure of large proportions where the heat of hydration increases the temperature considerably above normal.

9. The indications are that, under conditions ordinarily obtaining in mass concrete construction with a normal cement as at present manufactured and with normal temperatures at the time of mixing, there is little increase in strength after 28 days.

10. The temperature at which mass concrete is placed effects appreciably the strength, not alone at the early ages, but at the later ages as well. For the richer mixes in particular, a higher initial temperature leads to a lower strength at the later ages, though the reverse is true at the early ages.

# SPECIFICATIONS FOR HOLLOW MASONRY BUILDING UNITS[1]

By Douglas E. Parsons[2]

## Synopsis

From an analysis of published results of tests on large masonry specimens of clay hollow units, it was found that properties of the units not considered by current specifications had a marked influence on the compressive strength of the masonry. For end-construction masonry it appeared that the proportion of the gross sectional area of the masonry bedded in mortar was an important factor. In view of this it is suggested that specifications for hollow masonry units should take account of the features of the design of the units likely to have an influence on the proportion of their sectional areas given a mortar bearing in masonry.

The data pertaining to side-construction masonry indicate that the following geometrical properties of the units have an important effect on the strength of masonry and hence should be considered in formulating specifications: (1) ratio of thickness of horizontal bearing shells to maximum span between vertical supports (shells and webs), and (2) features of design affecting the proportion given a bearing at bed joints.

## Introduction

It has become a common practice in this country for some organizations or groups to prepare specifications for engineering materials while others formulate regulations or codes to govern the methods of using the materials in structures. This practice has many advantages and perhaps no serious disadvantages if the work of the different groups is coordinated to result in specifications and codes that will permit the most economical construction consistent with safety.

As changes in manufacturing and construction practices are constantly taking place it is necessary to examine periodically the current specifications and codes in order to be certain that they remain compatible with safety and economy in construction. Although there is perhaps no marked trend in that direction in the hollow concrete masonry industry, it has been observed that some units are being made with thinner shells than was common heretofore.[3] When hollow masonry units of clay were first introduced on a large scale, shells more than 1 in. thick were the rule, whereas recent production is confined almost entirely to units having shells less than 1 in. thick. No such decided change in concrete units is expected, but national specifications and codes should cover all of the extensively used types of hollow units.

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[1] Publication approved by the Director of the Bureau of Standards of the U. S. Department of Commerce.

[2] Chief, Masonry Construction Section, U. S. Bureau of Standards, Washington, D. C.

[3] D. R. Collins, "Achieving Light-Weight Building Units with Ordinary Aggregates," *Concrete*, Vol. 37, November, 1930, p. 20; also J. J. Buzzell, "Media Plant Pioneers in Manufacture of Light-Walled Units," *Concrete*, Vol. 38, March, 1931, p. 21.

The designs or types of hollow units undoubtedly have a bearing on many of the service characteristics of walls built of them and some of these should be considered in connection with the requirements of specifications and codes, but only the factors affecting the compressive strength of walls will be considered in this paper.

Working stresses (the maximum stresses that are considered safe) for building walls of hollow units usually are the same for units having equal compressive strengths. The magnitudes of working stresses specified in building codes appear to be based upon the assumption that the compressive strength alone of the units is an adequate measure of the effect of their characteristics on wall strengths. The distribution of the values shown in Figs. 1 and 5 resulting from tests of large masonry specimens indicates strongly that there are other properties of the units which have an important effect on the strength of masonry. It seemed worth while to study further the test data and to find, if possible, a better measure of masonry strength.

## TEST DATA

The reports of Talbot and Abrams,[1] Whittemore and Hathcock[2] and Stang, Parsons and Foster[3] give the results of tests on large masonry specimens of hollow clay units in which the arrangement of bonding of the units was similar to that in ordinary building construction. These reports contain also rather complete data on, and physical tests of, the mortars and units used together with descriptions of the methods of construction and the workmanship. No other data[4] on tests of large masonry specimens of either hollow clay or hollow concrete units with as complete information on materials and workman-

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[1] A. N. Talbot and Duff Abrams, "Tests of Brick Columns and Terra Cotta Block Columns," *Bulletin No. 27*, University of Illinois (1908).

[2] H. L. Whittemore and B. D. Hathcock, "Some Compressive Tests of Hollow-Tile Walls," U. S. Bureau of Standards *Technologic Paper No. 238* (1923).

[3] A. H. Stang, D. E. Parsons and H. D. Foster, "Compressive and Transverse Strength of Hollow-Tile Walls," U. S. Bureau of Standards *Technologic Paper No. 311* (1926).

[4] Data on the strength of walls 3 ft. long and 3 ft. high are reported by S. H. Ingberg and H. D. Foster in the paper "Fire Resistance of Hollow Load-Bearing Wall Tile," Research Paper No. 37, *Bureau of Standards Journal of Research*, Vol. 2 (1929). Tests of pilasters under eccentric loading are described and discussed by J. R. Shank and H. D. Foster in "Strength of Brick and Tile Pilasters Under Varied Eccentric Loading," *Bulletin No 51*, Ohio State University (1930).

Table I.—Compressive Strength of End-Construction Masonry of Hollow Units.

| Designation | Author's Specimen Numbers | Type of Unit and Size, in. | Mortar |  |  | Estimated Ratio of Bearing Area to Gross Area | Compressive Strength of Units, lb. per sq. in. |  | Compressive Strength of Masonry (gross area), lb. per sq. in. | Source of Data |
|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  | Mixture by Volume | Specimens | Compressive Strength, lb. per sq. in. |  | Gross Area | Net Area |  |  |
| No. 1...... | 1, 2 | 3-cell 4 by 8.5 by 8 (thick shells) | 1:2 Cement Mortar | ...... | .... | 0.85 | 3470 | 4100 | 2880 | a |
| No. 2...... | 3, 4 | 3-cell 4 by 8.5 by 8 (thick shells) | 1:2 Cement Mortar | ...... | .... | 0.85 | 3470 | 4100 | 3070 | a |
| No. 3...... | 5, 6 | 3-cell 4 by 8.5 by 8 (thick shells) | 1:2 Cement Mortar | ...... | .... | 0.74 | 3470 | 4100 | 2950 | a |
| No. 4...... | 8, 15 | 3-cell 4 by 8.5 by 8 (thick shells) | 1:3 Cement Mortar | 6-in. cubes | 3400 | 0.74 | 5170 | 6110 | 4300 | a |
| No. 5...... | 12 | 3-cell 4 by 8.5 by 8 (thick shells) | 1:3 Cement Mortar | 6-in. cubes | 3130 | 0.74 | 5170 | 6110 | 3300 | a |
| No. 6...... | 17 | 3-cell 4 by 8.5 by 8 (thick shells) | 1:3 Cement Mortar | 6-in. cubes | 3370 | 0.74 | 5170 | 6110 | 3050 | a |
| No. 7...... | 18, 19 | 3-cell 4 by 8.5 by 8 (thick shells) | 1:5 Cement Mortar | ...... | .... | 0.74 | 5170 | 6110 | 3350 | a |
| No. 8...... | 1, 2, 3 | XXX, 12 by 12 by 12 | 1:¼:3 Cement-Lime Mortar | 2-in. cubes | 2150 | 0.41 | 2850 | 6930 | 933 | b |
| No. 9...... | 4, 5, 6 | XXX, 6 by 12 by 12 | 1:¼:3 Cement-Lime Mortar | 2-in. cubes | 2150 | 0.47 | 4020 | 8520 | 1020 | b |
| No. 10...... | 7, 8, 9 | XXX, 8 by 12 by 12 | 1:¼:3 Cement-Lime Mortar | 2-in. cubes | 2150 | 0.46 | 2320 | 5010 | 1040 | b |
| No. 11...... | 25, 26 | XXX, 12 by 12 by 12 | 1:¼:3 Cement-Lime Mortar | 2-in. cubes | 2150 | 0.45 | 1680 | 3770 | 930 | b |
| No. 12...... | 27, 28 | XXX, 6 by 12 by 12 | 1:¼:3 Cement-Lime Mortar | 2-in. cubes | 2150 | 0.53 | 1900 | 3610 | 690 | b |
| No. 13...... | 1-E-1 | 6-cell, 8 by 12 by 12 | 1¼:3 Lime Mortar | 2 by 4-in. cylinders | 85 | 0.19 | 1830 | 4470 | 90 | c |
| No. 14...... | 1-E-2 | 6-cell, 8 by 12 by 12 | 1:1¼:6 Cement-Lime Mortar | 2 by 4-in. cylinders | 760 | 0.19 | 1830 | 4470 | 275 | c |
| No. 15...... | 4-E-2 | 6-cell, 8 by 12 by 12 | 1:1¼:6 Cement-Lime Mortar | 2 by 4-in. cylinders | 760 | 0.23 | 1270 | 2740 | 295 | c |
| No. 16...... | 6-E-2 | XXX, 8 by 12 by 12 | 1:1¼:6 Cement-Lime Mortar | 2 by 4-in. cylinders | 760 | 0.34 | 2100 | 4260 | 370 | c |
| No. 17...... | 2-E-2 | 6-cell, 12 by 12 by 12 | 1:1¼:6 Cement-Lime Mortar | 2 by 4-in. cylinders | 760 | 0.25 | 1860 | 4940 | 335 | c |
| No. 18...... | 1-E-3 | 6-cell, 8 by 12 by 12 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 0.26 | 1830 | 4470 | 460 | c |
| No. 19...... | 5-E-3 | 6-cell, 8 by 12 by 12 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 0.28 | 2840 | 6470 | 550 | c |
| No. 20...... | 6-E-3 | XXX, 8 by 12 by 12 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 0.49 | 2100 | 4260 | 650 | c |
| No. 21...... | 7-E-3 | XXX, 8 by 12 by 12 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 0.39 | 2030 | 5280 | 610 | c |
| No. 22...... | 9-E-3 | Double shell, 8 by 12 by 5 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 0.30 | 3320 | 6900 | 555 | c |
| No. 23...... | 1-E-4 | 6-cell, 8 by 12 by 12 | 1:3 Cement Mortar | 2 by 4-in. cylinders | 1990 | 0.19 | 1830 | 4470 | 355 | c |

[a] *Bulletin No. 27*, University of Illinois (1908).
[b] U. S. Bureau of Standards *Technologic Paper No. 238* (1923).
[c] U. S. Bureau of Standards *Technologic Paper No. 311* (1926).

TABLE II.—COMPRESSIVE STRENGTH OF SIDE-CONSTRUCTION MASONRY OF HOLLOW UNITS.

| Designation | Author's Specimen Numbers | Type of Unit and Size, in. | Mortar |  |  | Compressive Strength of Units, lb. per sq. in. |  | Compressive Strength of Masonry (gross area), lb. per sq. in. | Source of Data |
|---|---|---|---|---|---|---|---|---|---|
|  |  |  | Mixture by Volume | Specimens | Compressive Strength, lb. per sq. in. | Gross Area | Net Area |  |  |
| No. 24........ | 10, 11, 12 | XXX, 12 by 12 by 12 | 1:¼:3 Cement-Lime Mortar | 2-in. cubes | 2150 | 840 | 3780 | 375 | a |
| No. 25........ | 13, 14, 15 | XXX, 8 by 12 by 12 | 1:¼:3 Cement-Lime Mortar | 2-in. cubes | 2150 | 1240 | 5460 | 395 | a |
| No. 26........ | 16, 17, 18 | XXX, 6 by 12 by 12 | 1:¼:3 Cement-Lime Mortar | 2-in. cubes | 2150 | 1665 | 5930 | 700 | a |
| No. 27........ | 29, 30 | XXX, 6 by 12 by 12 | 1:¼:3 Cement-Lime Mortar | 2-in. cubes | 2150 | 585 | 1870 | 380 | a |
| No. 28........ | 1-S-1 | 6-cell, 8 by 12 by 12 | 1¼:3 Lime Mortar | 2 by 4-in. cylinders | 85 | 500 | 2210 | 165 | b |
| No. 29........ | 1-S-2 | 6-cell, 8 by 12 by 12 | 1:1¼:6 Cement-Lime Mortar | 2 by 4-in. cylinders | 760 | 500 | 2210 | 285 | b |
| No. 30........ | 8-S-2 | H-shaped, 8 by 10¼ by 12 | 1:1¼:6 Cement-Lime Mortar | 2 by 4-in. cylinders | 760 | 1040 | 3350 | 455 | b |
| No. 31........ | 2-S-2 | 6-cell, 12 by 12 by 12 | 1:1¼:6 Cement-Lime Mortar | 2 by 4-in. cylinders | 760 | 580 | 2620 | 385 | b |
| No. 32........ | 1-S-3 | 6-cell, 8 by 12 by 12 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 500 | 2210 | 405 | b |
| No. 33........ | 4-S-3 | 6-cell, 8 by 12 by 12 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 330 | 1290 | 230 | b |
| No. 34........ | 5-S-3 | 6-cell, 8 by 12 by 12 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 1740 | 7080 | 500 | b |
| No. 35........ | 6-S-3 | XXX, 8 by 12 by 12 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 430 | 1500 | 240 | b |
| No. 36........ | 10-S-3 | 2-cell, 8 by 5 by 12 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 820 | 3740 | 335 | b |
| No. 37........ | 13-S-3 | 3-cell, 8 by 5 by 12 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 900 | 3540 | 440 | b |
| No. 38........ | 14-S-3 | T-shaped, 8 by 6¼ by 12 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 340 | 1150 | 305 | b |
| No. 39........ | 15-S-3 | T-shaped, 8 by 6¼ by 12 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 630 | 1870 | 430 | b |
| No. 40........ | 14-S-3 | T-shaped, 8 by 6¼ by 12 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 340 | 1150 | 340 | b |
| No. 41........ | 16-S-3 | Cube, 8 by 8 by 8 | 1:1¼:4 Cement-Lime Mortar | 2 by 4-in. cylinders | 1190 | 780 | 3060 | 430 | c |
| No. 42........ | 1-S-4 | 6-cell, 8 by 12 by 12 | 1:3 Cement Mortar | 2 by 4-in. cylinders | 1990 | 500 | 2210 | 360 | b |

[a] U. S. Bureau of Standards *Technologic Paper No. 238* (1923).
[b] U. S. Bureau of Standards *Technologic Paper No. 311* (1926).
[c] Unpublished data.

ship have come to the author's attention. Indeed there appears to be no published information of this character on the strength of masonry of hollow concrete units.[1]

A summary of the data pertaining to compression tests under concentric loading of end-construction masonry (cells vertical) with broken joints is given in Table I. A similar summary of the data pertaining to side-construction masonry (cells horizontal) is given in Table II.

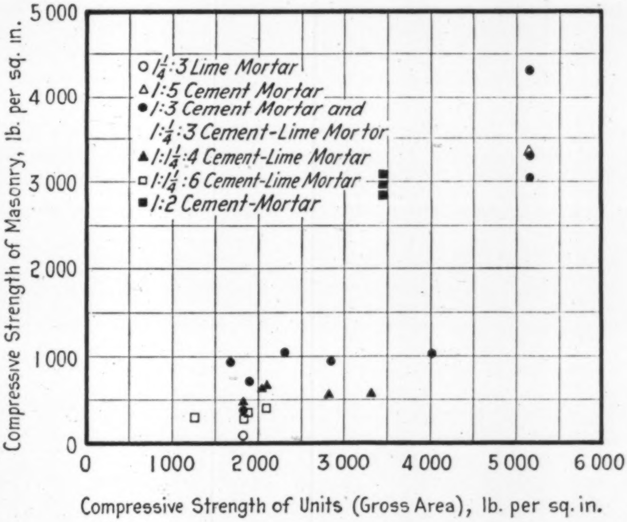

FIG. 1.—Relation Between Compressive Strength (Gross Area) of End-Construction Masonry and Compressive Strength of Units.

The proportions for the mortars are by volume.

Values of the strength of the mortar specimens are lacking in a few cases as indicated in Table I. And the strength (2150 lb. per sq. in.) given for the 1:¼:3 mortar[2] at the age for testing the masonry was estimated from the strengths at the ages of 7 days and 6 months.

The ratios in Table I of bearing area to gross area of masonry are estimates based upon the dimensions of the units and descriptions (and observations in the case of the tests reported by the Bureau of Standards)[3] of the workmanship and method of laying the units.

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[1] Since this was written, the following papers have appeared: C. A. Menzel, "Tests of the Fire Resistance and Stability of Walls of Concrete Masonry Units," see p. 607; and F. E. Richart, P. M. Woodworth and R. B. B. Moorman, "Tests of the Stability of Concrete Masonry Walls," see p. 661.

[2] Proportions by volumes, respectively, of cement, lime and sand.

[3] U. S. Bureau of Standards *Technologic Paper No. 311*,

## RELATIONS BETWEEN PROPERTIES OF THE MASONRY AND THE UNITS

### *End-Construction Masonry:*

By comparing the distributions of the values in Figs. 1 and 2, it appears that the ratio of the bearing area to the gross area has an important effect on the strength of the masonry. Without going into detail regarding the method[1] used, it was found that the following form of empirical equation fits the data for the end-construction masonry as well as any other equally simple form investigated:

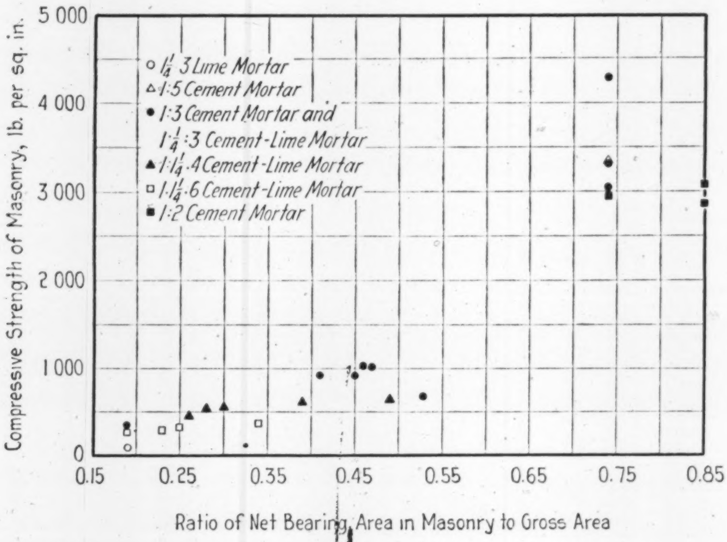

FIG. 2.—Relation Between Compressive Strength (Gross Area) of End-Construction Masonry and the Ratio of Net Bearing Area in Masonry to Gross Area.

The proportions for the mortars are by volume.

$$M = Kb\sqrt{mu} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where $M$ = compressive strength of masonry (gross area), in pounds per square inch;

$K$ = a numerical coefficient, approximately equal to unity;

$b$ = ratio of area in bearing to gross sectional area of masonry;

$u$ = compressive strength of the units (gross area), in pounds per square inch; and

$m$ = compressive strength of mortar specimens aged the same as the masonry, in pounds per square inch.

Figure 3 shows the relation between strengths of masonry found from tests and those calculated by Eq. 1 with $K = 1$. The scattering of the plotted points gives some indication that other factors than those included in the equation may affect the strength of masonry, as might be expected. However, part of the scattering may be attributed rather definitely to known causes. For example, the two values (for Nos. 9 and 12, Table I) for which the ratio of test to estimated strength are lowest pertain to specimens 12 ft. high and only 6 in. thick. It seems likely that the large ratio of height to thickness of the walls accounts in part for the unusually low strengths of these specimens. Of the three values (for Nos. 4, 5 and 6, Table I) from the data of Talbot and Abrams, one pertains to masonry specimens built with units specially selected because of their uniformity of size and shape, another to units having warped bearing surfaces, and the third to a "poorly laid" masonry specimen. Obviously such factors as these are not taken into account by Eq. 1.

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[1] Developed by Edgar Buckingham, "Model Experiments and the Forms of Empirical Equations," *Transactions*, Am. Soc. Mechanical Engrs., Vol. 37, p. 263 (1915).

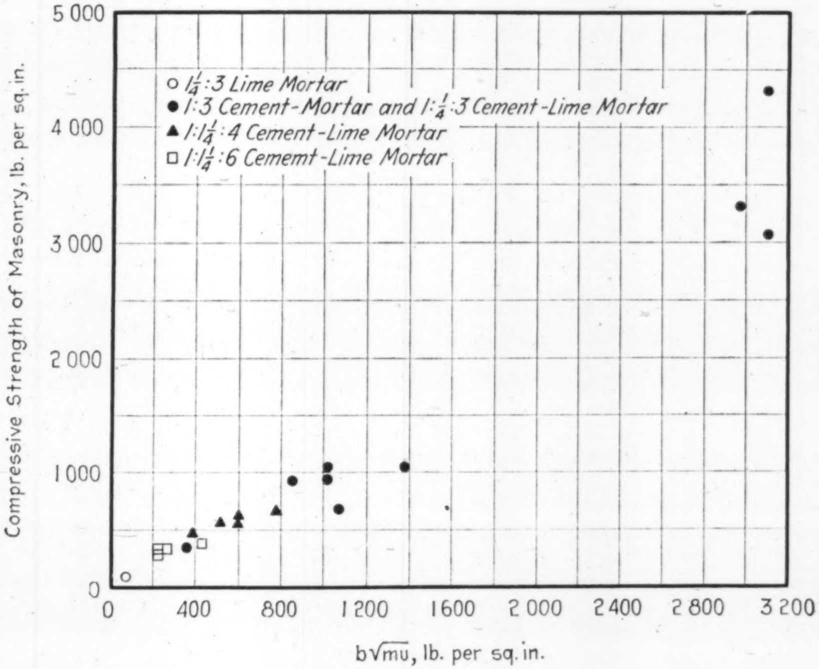

FIG. 3.—Relation Between Compressive Strength (Gross Area) of End-Construction Masonry and $b\sqrt{mu}$.

$b$ = ratio of bearing area in masonry to gross area;
$m$ = compressive strength of mortar specimens, pounds per square inch; and
$u$ = compressive strength (gross area) of units, pounds per square inch.
The proportions for the mortars are by volume.

This equation is incomplete in other respects and must be considered, therefore, as only a rough approximation. Any factors having an influence on the strength of the masonry may be the cause of some of the scattering of the plotted points in Fig. 3, unless they are included in Eq. 1. The following are the most important of those omitted:

1. The elastic properties of the mortars and units;
2. Such geometrical properties as the size and shape of the masonry, thickness of the mortar joints and regularity of the bearing surfaces of the units;

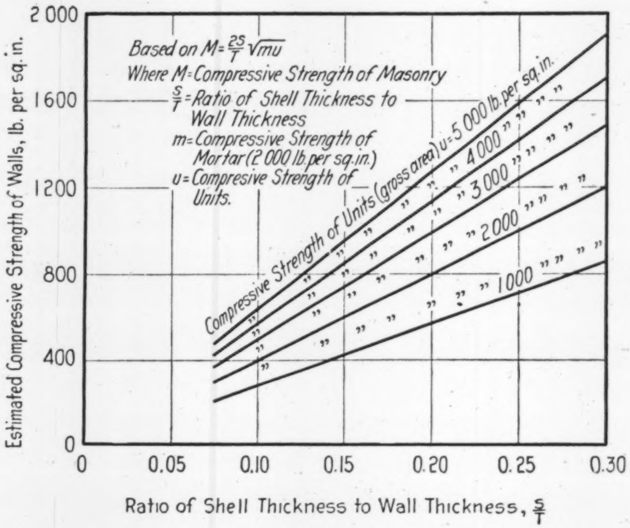

FIG. 4.—Estimated Strengths of End-Construction Walls of Hollow Masonry Units (for 1:3 cement mortar, assumed strength 2000 lb. per sq. in.).

In estimating the strength of walls built with other mortars, the following percentages of the values shown for the 1:3 cement mortar may be used: with 1:1:4 cement-lime mortar, 75 per cent and with 1:1:6 cement-lime mortar, 60 per cent.

3. Differences between the physical properties of the mortar in the masonry and those of the mortar specimens; and
4. Details of the testing such as capping materials and procedure and rate of loading.

Aside from the incompleteness of Eq. 1, the data of Table I on which the equation was based is deficient in that values for masonry composed of thick-shelled units of low compressive strength are not included.

It is convenient, nevertheless, to have available a method for making rough estimates of expected masonry strengths with different types of units, particularly with those of the end-construction type because of the wide difference in the thickness of shells. The test data indicate that properties of the units other than their compressive strengths must be considered for even rough estimates of the compressive strength of end-construction masonry. If Eq. 1 is to be used for this purpose, it is necessary to estimate the values of $b$ and $m$. With some types of units, and probably with all in some localities, it is common practice to provide a mortar bearing only for the exterior or face shells forming the wall surfaces and for the end shells at the points where they cross the interior webs of the units in the course next below. Many of the concrete units are designed to provide a bearing only on the exterior shells forming the wall surfaces. Hence, for conservative estimates of masonry strength, the value of $b$ should be calculated on this basis unless the workmanship is known to be different.

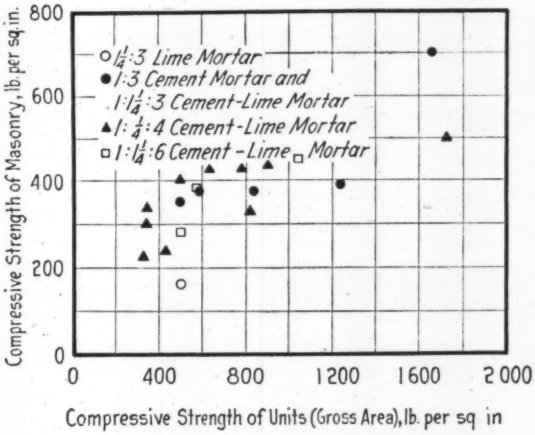

FIG. 5.—Relation Between Compressive Strength (Gross Area) of Side-Construction Masonry and Compressive Strength of Units.

The proportions for the mortars are by volume.

The values shown in Fig. 4 are based upon Eq. 1 for $K = 1$ and

$$\text{for } b = \frac{2\,S}{T},$$

where $S$ = thickness of shells forming the wall surfaces, in inches; and
$T$ = thickness of wall, in inches.

*Side-Construction Masonry:*

The relation between the compressive strengths (gross area) of side-construction masonry and of the units is shown in Fig. 5. Here the relation seems to be somewhat more regular than for end-construction masonry illustrated in Fig. 1.

A part of the lack of regularity in the distribution of the plotted points in Fig. 5 is assignable to known causes. The masonry specimens designated Nos. 33 and 35 in Table II were built with units having the central portions of the bearing surfaces recessed. With such units, only those portions of the horizontal shells adjacent to the outer or vertical shells provided a bearing. It is understood that these tiles were formed in this manner purposely in order to reduce the probability of continuous mortar joints through walls. For ordinary tile of this design, about 70 per cent of the net sectional area would be given a direct bearing.

Differences in the ratios between the thickness of the bearing (or horizontal) shells and the horizontal distances between the supports provided by the vertical shells and webs probably is another major cause for the lack of a regular relation in Fig. 5. Comparing No. 36 with No. 37 in Table II, it is seen that the masonry with 3-cell tiles was stronger than that with the 2-cell units. In fact, all of those having relatively closely spaced vertical shells and webs (Nos. 30, 37, 38, 39 and 40) lie above an average line through the points of Fig. 5. It has been observed [1] that in many cases the cracking of the horizontal shells in contact with the mortar was the first sign of the failure of side-construction walls in compression tests. It seems apparent, therefore, that the bending strength of the shells is important with some unit designs.

However, the available data relating to side-construction masonry are inadequate for a satisfactory determination of the form of an empirical equation representing the relations between the strength of masonry and the properties of the units. In fact the two important factors mentioned in the preceding paragraphs cannot be evaluated at present.

For the end-construction masonry the ratio of area in bearing to gross sectional area of masonry is a function of the design of the units and the workmanship, as measured by the proportion of the shells and webs given a mortar bearing by the mason. No such simple way of evaluating the effect of workmanship in side-construction walls is apparent, although it is probable that there were marked differences in the workmanship of the different masons. Walls Nos. 32 and 42 of Table II were of identical construction except for the mortar and workmanship. The tests by Ingberg and Foster[2] gave, on the average, higher strengths for walls with cement mortar than for those with 1:1:4 cement-lime mortar. Hence, it is logical to assume that with the same workmanship, No. 42 would have been stronger instead of weaker than No. 32. The strength (405 lb. per sq. in.) of No. 32 with 1:1¼:4 cement-lime mortar was appreciably greater than the strength (360 lb. per sq. in.) of No. 42 with the much stronger 1:3 cement mortar, indicating that the workmanship for No. 32 was superior to that for No. 42.

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[1] U. S. Bureau of Standards *Technologic Paper No. 311*, p. 339.

[2] See Table 11, Research Paper No. 37 *Bureau of Standards, Journal of Research*, Vol. 2 (1929).

Without some way of evaluating the effects of workmanship, direct comparisons to determine the relations between wall strength and properties of the mortars are not easily made. About the only general statement warranted is that the use of stronger mortars usually results in stronger masonry.

In so far as the units are concerned, the data indicate that the three following properties of the units have an important effect on the strength of side-construction masonry: (1) compressive strength, (2) ratio of thickness of bearing shells to maximum span between vertical supports (shells and webs), and (3) feature of design affecting the proportion of area given a bearing at bed joints.

## APPLICATION TO SPECIFICATIONS

The question naturally arises whether specifications for hollow masonry units should be such that products of one class, as defined by specifications, will provide, with all designs of units of that class, approximately equal masonry strengths; or whether building codes should make provisions for different working stresses depending on the design of the units. The answer should, of course, take into consideration economic factors which cannot be considered here.

Disregarding economic factors, and assuming that all detailed requirements for units should be included in specifications for units, it seems essential from the test data that such geometrical properties as shell thickness be included in such specifications. In fact, these properties appear to have a greater effect on the strength of end-construction masonry than any of the other properties of the units investigated.

In a practical way, the weight requirements of the Society's Standard Specifications and Tests for Hollow Burned-Clay Load-Bearing Wall Tile (C 34 - 30)[1] provide limits for shell thickness, because the weight per unit of bulk volume of the fired clay does not vary over an extremely wide range, such variations as exist being due largely to variations in porosity. Since the porosity, as measured by the water absorption of the tile, is roughly related to the weight per unit volume of the material,[1] a limitation on the permissible absorption range also tends to limit the permissible variation in this respect. Thus when building codes require for load-bearing construction the A.S.T.M. "hard" and "medium" classes of tile covering an average absorption range of from 6 to 16 per cent, the weight of the material is limited to a value within the range 140 to 115 lb. per cu. ft.

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[1] 1930 Book of A.S.T.M. Standards, Part II, p. 272.

A limitation on the gross weights of concrete units, however, would not serve the purpose of limiting shell thicknesses because of the wide variations in the weight per unit of bulk volume of concrete. By the use of different types of aggregates, the weights of the concrete in hollow units may be made to vary from 70 to 150 lb. per cu. ft. It appears, therefore, that if limitations on the shell thicknesses of concrete units are desired, they must be made directly rather than through weight requirements.

For side-construction masonry, it appears that the following geometrical properties of the units should be given consideration in the preparation of specifications for hollow masonry building units: (1) the ratio of thickness of bearing shells to maximum span between vertical supporting shells (and webs), and (2) the features of design of the units affecting the area given a mortar bearing.

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[1] Report of Committee C-10 on Hollow Tile, *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part I, p. 415 (1924).

[For Discussion on Masonry Building Units, see page 681.—ED.]

# TESTS OF THE FIRE RESISTANCE AND STABILITY OF WALLS OF CONCRETE MASONRY UNITS

By C. A. Menzel[1]

## Synopsis

This paper presents the principal results thus far obtained in a comprehensive and systematic investigation of the fire resistant and load-carrying properties of approximately 100 walls of concrete masonry units when subjected to standard fire endurance and load tests. The tests covered studies of the relative influence of such factors as type and grading of aggregate, cement content, design of unit, type of mortar, workmanship, and application of plaster.

With the exception of size of wall, the tests were conducted strictly in accordance with Standard Specifications for Fire Tests of Building Construction and Materials. Both because of limitations in the space available for the test equipment and the excessive cost in a series of tests of this magnitude, walls about 5½ ft. wide and 6 ft. high with a total area of about 33 sq. ft. were employed instead of walls 9 ft. in height and 100 sq. ft. in area required by these specifications. To compensate for the reduction in size of the test walls as well as to improve the precision with which such tests are usually conducted, a particularly rigid test technique was adopted. It is believed that the technique followed gives a reliable basis for comparison of the relative fire-resistant properties of the walls and of the relative influence of the various factors studied.

Information is presented on the load-carrying ability of the walls before and after severe fire exposure for a wide range in composition and design of the units and of the type of mortar and workmanship employed in the construction of the walls.

## Introduction

Knowledge of the performance of walls, floors, columns, and other structural building members upon exposure to fire is of major importance in designing constructions which are safe and which are not a menace to neighboring structures nor to the public. Recognition of this is registered in the many codes of regulatory bodies. It is important to secure balance between the many members in a single building, and of buildings of like character and use in a community and also to secure uniformity in the requirements of various communities throughout the country. To do this it is necessary that the fire-resistive properties of materials and assemblies be measured and specified according to a common standard expressed in terms which are applicable alike to a wide variety of materials, situations and conditions of exposure.

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Some valuable information regarding the performance of walls of concrete masonry units when subjected to standard fire endurance, water stream and excess load tests has been obtained from tests conducted from time to time since 1921 at Underwriters' Laboratories, Chicago. It was apparent from these tests that a comprehensive investigation of the relative influence of the various factors involved would not only provide definite information of great value to architects, building officials, insurance companies and others interested, but would also establish the underlying principles governing the fire resistance and load-carrying properties of walls of concrete masonry units.

Preparations for such an investigation were started in June, 1928, with the design and construction of suitable equipment for the conduct of standard fire tests in the Research Laboratory of the Portland Cement Association, Chicago, and the installation of a block machine and mixer for the manufacture of the units.

This paper gives a brief description of the apparatus and test procedure and discusses the principal results thus far obtained in tests on more than 100 walls representing approximately two-thirds of the test work scheduled for the investigation. A comprehensive and detailed report will be prepared covering the entire investigation at the conclusion of the test program.

## SCOPE OF TESTS

The tests were arranged to study the relative influence of the various factors of composition, form, and assembly of the units on the performance of the walls when subjected to standard fire endurance and load tests. The following factors were included: (1) grading of aggregate, (2) cement content, (3) type of aggregate, (4) design of unit, (5) filling hollow walls with various materials, (6) application of plaster, (7) type of mortar, (8) workmanship in laying walls, (9) time of curing, (10) coal and coke in cinder aggregate, (11) mixture of two widely different aggregates, and (12) furnace temperatures below and above standard.

In general, the tests were scheduled in groups to bring out the effect of each factor through a wide range of variations in that factor. As the work progressed it was found desirable to include several miscellaneous tests to obtain information on the significance of certain combinations of factors. About one-fourth of the panels were built in pairs to determine the strength of the walls both before and after fire exposure.

The fire endurance and load tests were supplemented by other tests and examinations to determine the composition and properties of the materials and such physical properties of the individual units as weight, absorption and compressive strength.

## DESCRIPTION OF EQUIPMENT FOR FIRE AND LOAD TESTS

*Apparatus for Mounting, Loading and Handling Test Walls.*—The equipment was designed specifically for the conduct of fire tests in accordance with the Standard Specifications for Fire Tests of Building Construction and Materials.[1] Lack of both head and side room made it necessary to confine the capacity of the test equipment to wall specimens $5\frac{1}{2}$ ft. wide by 6 ft. high with an area of 33 sq. ft. as compared with requirements for standard wall specimens of 9 ft. height and 100 sq. ft. area. The selection and installation of apparatus was based on considerations of adaptability for controlling and duplicating test conditions and for making accurate observations as well as on considerations of flexibility, ease of manipulation and cost.

The general design of this apparatus and its scheme of operation may best be seen from Fig. 1. The elevation at the right illustrates the general assembly and the relation of the various parts as viewed from the side of the test wall not exposed to fire. The cross-sectional views at the left and at the top show further details.

The various pieces of apparatus are so arranged that the wall *F* can be installed in the opening of a movable steel test frame, *A*, loaded by means of four hydraulic jacks, *D*, and exposed on one face to the flames of a vertical, gas-fired furnace. After fire exposure, the test wall can immediately be withdrawn from the furnace either for cooling in freely circulating air, or for exposure to the impact, cooling, and eroding action of a standard water stream applied to the incandescent face. The steel frame is hung from trolleys *B*, running on the bridge of an overhead crane *C* and is so designed that walls 8 in. thick and $5\frac{1}{2}$ ft. wide can be loaded to over 700 lb. per sq. in. of gross bearing area or to a total of 400,000 lb. The load applied by the hydraulic jacks is controlled by controlling the liquid pressure supplied to them by a hand-operated high pressure pump through the flexible hose *H* and manifold *G*. Several high-pressure air cylinders of sufficient capacity connected to the hydraulic system provide an air cushion and storage chamber, which makes it possible to keep the test panels under practically constant load for long periods of time regardless of expansion or contraction.

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[1] Tentative American Standard A2–1926. Prepared by the Sectional Committee on Fire Test Specifications, under the joint sponsorship of the U. S. Bureau of Standards, the American Standards Association Fire Protection Group, and the American Society for Testing Materials, functioning under the procedure of the American Standards Association (formerly A.E.S.C.). These specifications are a Tentative Standard of the American Society for Testing Materials, Serial Designation: C 19 – 26 T. They were adopted in May, 1926, as a Standard of the National Fire Protection Association and by the Fire Council of Underwriters' Laboratories.

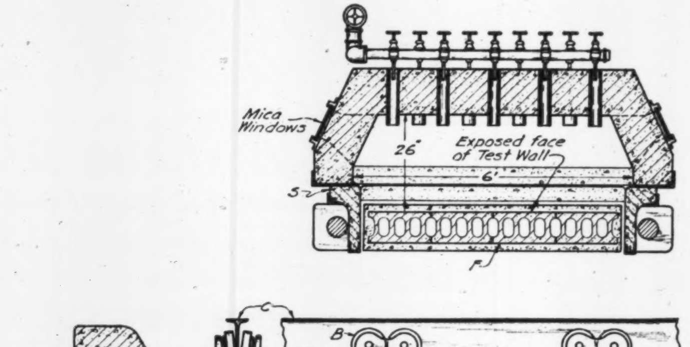


Fig. 1.—General Arrangement of Test Furnace and Movable Test Frame.

*Test Furnace.*—The furnace consists of a shallow vertical combustion chamber with back, sides and roof of portland-cement concrete about 12 in. thick made with crushed fire brick as fine and coarse aggregate. The side walls are placed at an angle with the back wall and each is provided with three observation holes glazed with mica so that all parts of the interior of the combustion chamber and the exposed face of the test wall may be observed. The products of combustion leave through the roof of the furnace and thence to the stack of the building.

The furnace is heated by a bank of 27 gas burners passing through the back wall in six horizontal rows 12 in. apart. To promote a good distribution of the fire over the exposed surface of the test wall, the burners are staggered by placing alternately 4 and 5 burners in the adjacent horizontal rows. Individual valves on each burner permit close control of the gas and flame. Most of the air required for combustion is drawn by natural draft through 5 holes 4 in. in diameter at the bottom of the combustion chamber. Just enough air is drawn in at each burner by jet action and premixed with the gas to bring the flame into sweeping contact with the panel without impingement. A quiet, bathing luminous fire, free from smoke, is obtained by controlling the draft by a damper in the 12-in. flue pipe leading to the stack. A sensitive draft gage connected to the top and bottom of the combustion chamber enables the operator to maintain uniform draft conditions.

A feature of the design of this furnace is the absolute elimination of all iron from the combustion chamber in the form of burner tubes, stay rods, etc. This precaution was taken to prevent the formation of iron oxide which might affect the thermo-electric properties and, hence, the accuracy of the thermocouples which are used to indicate the temperature in the furnace.

*Measurement of Temperature, Expansion and Deflection.*—The furnace fire is controlled by manually operating the gas valves. By a system of sensitive thermocouples the temperatures are made to follow the standard furnace control curve shown in Fig. 4. Other thermocouples are provided for measuring temperatures at various stations within the wall and on the unexposed face. Standard partial-immersion thermometers also are used to measure temperatures under standard asbestos pads cemented to the unexposed face of the walls. Changes in vertical dimensions of the walls during and after exposure to load and fire are measured by sensitive gages, *K*, Fig. 1. A weighted steel wire stretched about 4 in. from the unexposed face near its vertical center line provides a reference base for measurements of the deflection or bowing of the wall during test. A clearance of from 1 to 2 in. is allowed between the vertical edges of the test wall and the concrete insulating the vertical frame members so that the loaded wall is free to expand and deflect in a horizontal direction during and after exposure to fire. To prevent flame passage, this clearance is closed with light, fluffy mineral wool or other suitable insulating material. The wall is restrained vertically by keeping the hydraulic jacks balanced at the normal safe working load during and after exposure to fire. Each test wall is built upon and supported by an insulated steel loading beam, *E*, and when mounted in the test frame completely fills the vertical opening between this beam and the upper horizontal member of frame

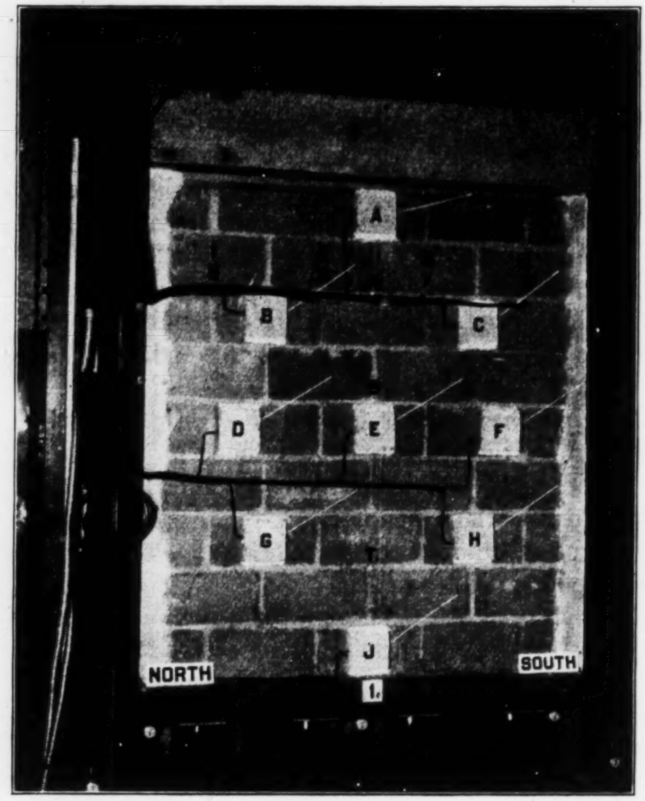


FIG. 2.—Showing a Wall Installed Within the Movable Steel Frame Ready for Test.

*A*, which was insulated with concrete, *S*. Figure 2 shows a wall installed within the moveable steel frame ready for test.

## TEST PROCEDURE

The various steps in the routine test procedure may be briefly described as follows:

(*a*) *Examination and Preparation of Aggregate.*—The fine and coarse aggregates were examined as to their general appearance, grading and other physical properties, and mineral composition. When necessary they were dried and screened to the desired sizes.

(*b*) *Molding and Curing Concrete Units.*—To control and facilitate the making of the units and the construction of the wall specimens in a regular and routine manner, a block machine and mixer of standard make were installed. The block machine was provided with 6 sets of mold boxes and tampers with which the typical designs of units illustrated in Figs. 8 and 9 were made.

Each set of units was molded from a carefully proportioned weight mixture of aggregate, cement, and water. The portland cement was a mixture of four brands purchased in Chicago. The correct quantities of aggregate and cement were first mixed dry for 1 minute. As much water as possible was then added to the dry mixture but not so great an amount as would appreciably impair the handling properties of the units during and after molding. The consistency was therefore, as wet as practicable for units made by the "tamped" process. The molds were filled in 2 layers, the amount of tamping of each layer being maintained constant for each design of unit.

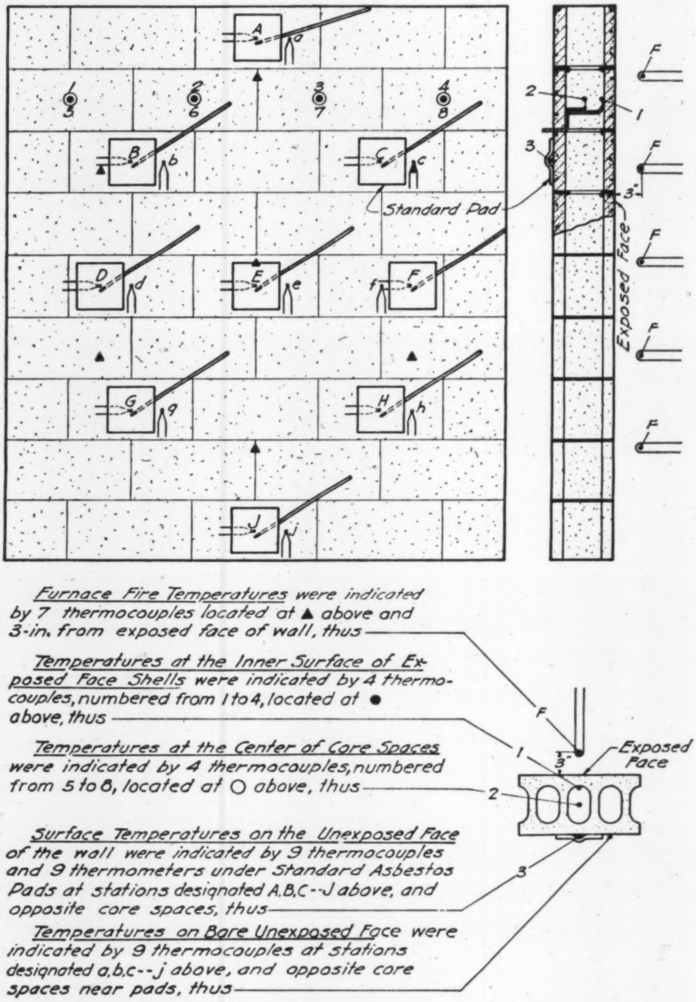
Immediately after molding, the units were removed to a room where they were "moist cured" for 5 days at a temperature of 70° F. and in a saturate foggy atmosphere obtained by atomizing nozzles. During the 5-day period a slight increase in weight due to the absorption of moisture was recorded. Upon removal from the moist room, the blocks were stored 9 days in ordinary air of the laboratory during which time they lost weight rapidly. They were laid up into test walls at age of 2 weeks.

(*c*) *Determination of Physical Properties of Units.*—The compressive strength of the units, both dry and in a saturated condition, was determined at 5, 28 and 60 days. The units were weighed periodically to determine the rate of moisture loss. Final weights in an air-dry and oven-dry condition were obtained as well as the water absorptive properties of the units.

(*d*) *Construction of Test Wall.*—When two weeks old, the units were laid up into test walls by one or the other of two workmen in the regular employ of the laboratory. Each panel was built upon a slab of reinforced concrete about 4 in. thick resting on a steel plate 2 in. thick, 12 in. wide, and 67 in. long. This "loading beam" was the medium by which the combined thrust exerted by the four hydraulic jacks was distributed to the wall after installation in the opening of the test frame of the furnace.

Except in certain special tests, the mortar used was a 1:3 portland-cement mixture with an addition of 15 per cent hydrated lime by volume of cement. The aggregate was Elgin sand (approximately half siliceous and half calcareous material) graded 0 to No. 14 having a fineness modulus of 1.90 to 2.00. This 1:3 volume mix was based on dry rodded sand and corresponds approximately to a 1:4 mix proportioned in the field with damp loose sand. In general, only as much mortar was employed in the horizontal joints of the block and tile as was required for the proper bedding of the face shells and intermediate parallel web, if any. This method of bedding is that designated as "good" in Fig. 13 (*a*) and Fig. 12 (*b*). The ends of adjacent units were in general bonded by hollow vertical mortar joints consisting of two strips of mortar extending inward 1½ to 2 in. from each face of the wall, leaving an air space between the strips. Both vertical and horizontal mortar joints averaged ¼ to ⅜ in. in thickness. The vertical core holes in the units and the hollow mortar joints formed in each wall assembly a series of interconnected vertical and horizontal hollow spaces, through which air and moisture could circulate and escape during the seasoning period.

Thermocouples for the measurement of temperatures in the interior of the test walls were carefully installed at the desired stations as the walls were laid up. The locations of these couples for walls of 3-oval-core block are shown in Fig. 3.



*Furnace Fire Temperatures* were indicated by 7 thermocouples located at ▲ above and 3-in. from exposed face of wall, thus

*Temperatures at the Inner Surface of Exposed Face Shells* were indicated by 4 thermocouples, numbered from 1 to 4, located at ● above, thus

*Temperatures at the Center of Core Spaces* were indicated by 4 thermocouples, numbered from 5 to 8, located at ○ above, thus

*Surface Temperatures on the Unexposed Face* of the wall were indicated by 9 thermocouples and 9 thermometers under Standard Asbestos Pads at stations designated A, B, C--J above, and opposite core spaces, thus

*Temperatures on Bare Unexposed Face* were indicated by 9 thermocouples at stations designated a, b, c--j above, and opposite core spaces near pads, thus

FIG. 3.—Diagram of Typical Thermocouple and Thermometer Locations for Walls of 3-Oval-Core Block.

(*e*) *Storing and Aging Wall.*—When the mortar had sufficiently set, each wall was moved into a storage space for drying and aging in freely circulating air. Although conditions for drying were quite favorable, all walls were aged at least 45 days during the heating season and 60 days during the summer months so as to approach as nearly as practicable the moisture content of the interior walls in service in heated buildings.

(*f*) *Installation of Wall in Test Frame of Furnace.*—By means of a hoist and transfer apparatus, the panels were installed within the opening of the movable steel test frame of the furnace five days before exposure to fire. The top bearing edge of the wall was buttered with mortar and caused to bear against the concrete lintel forming the insulation protecting the upper horizontal member of the test frame from the fire. The clearance between the ends of the wall and frame was loosely filled with mineral wool and magnesia which was covered with a thin layer of gypsum mortar to prevent flame passage.

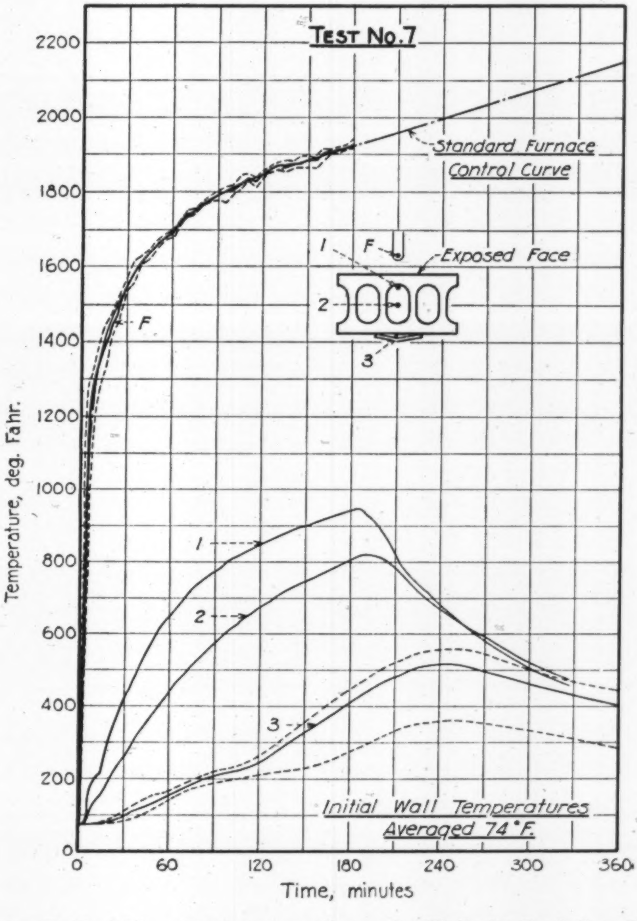


FIG. 4.—Typical Time-Temperature Curves for Wall of 3-Oval-Core Block.

Average temperatures are represented by full line curves; maximum and minimum temperatures by dotted line curves.

(*g*) *Mounting of Instruments for Temperature and Deflection Measurements.*—Thermocouples and thermometers were installed at various stations on the unexposed surface. Sensitive gages to register changes in vertical dimensions of the wall during test were mounted between the loading beam and the lower horizontal member of the steel test frame and along the vertical members. A fine steel wire was stretched vertically at the center of the wall as a reference base for measurements of the deflection or bowing.

(*h*) *Loading of Walls Before Fire Exposure.*—Just prior to fire exposure, a load of 240 lb. per sq. in. of gross wall area was gradually applied to each wall in approximately equal increments. This load is three times the normal working load of 80 lb. per sq. in.[1] required to be carried during the fire endurance test. After each load increment, observations were made of deflections of the wall and of the vertical deformations. Soon after these data were recorded, the load was reduced from 240 to 80 lb. per sq. in. gross area.

(*i*) *Intensity and Period of Fire Exposure.*—The wall, loaded to 80 lb. per sq. in. gross area, was exposed on one side to gas flames of controlled extent and intensity in accordance with a predetermined time-temperature curve shown in Fig. 4, designated in this paper as the "standard furnace control curve." It is identical with the time-temperature curve required by the standard fire test specifications. The fire exposure period was generally either 3 or 3½ hours for 8-in. walls, 5 hours for 12-in. walls, and 1¼ hours for 4-in. walls.

(*j*) *Measurement of Furnace Temperatures.*—Furnace temperatures were indicated by means of seven calibrated thermocouples symmetrically placed at various stations of the exposed wall area as indicated in Fig. 3. The temperature sensitive point of each couple was placed at a uniform distance of 3 in. from the exposed face of the test wall. The couples consisted of No. 8 gage, chromel-alumel wire twisted and welded at the sensitive tips. They were all enclosed in sealed "sillimanite" or "mullite" porcelain tubes 1 in. in outside diameter and ⅛ in. in wall thickness. The exposed length of the tube and couple in the furnace chamber was not less than 21 in. nor more than 25 in., depending on the thickness of wall.

(*k*) *Measurement of Temperatures Within Wall.*—In both the tile and block walls, temperatures within the wall were measured by means of thermocouples, consisting of No. 14 B. & S. gage, calibrated iron-constantan wire, covered with asbestos insulation and twisted and welded at the sensitive tips.

In the block walls, these thermocouples were arranged in the second course below the top and placed to give the temperatures at the inner surface of the exposed face shell and at the center of the core spaces. In general, not less than 8 thermocouples were symmetrically distributed over the width of the wall in this second course.

In the case of the tile walls, the thermocouples were arranged in the third course from the top to give the temperatures at the inner surface of the exposed face shell and at the corresponding surface of the central web. In general, not less than 10 thermocouples were distributed symmetrically over the width of the wall in this third course.

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[1] This load is prescribed in Section 11, page 10, of the "Recommended Minimum Requirements for Masonry Wall Construction," June 26, 1924, prepared by the Building Code Committee of the Dept. of Commerce which reads as follows: "*Working Stresses.*—The maximum allowable compressive stresses in masonry of hollow tile, concrete block, or concrete tile, or hollow walls of brick, due to combined live and dead loads, shall not exceed 80 lb. per sq. in. of gross sectional area, when laid in portland-cement mortar and 70 lb. per sq. in. of gross sectional area when laid with special cement-lime mortar."

(*l*) *Measurement of Temperatures of Unexposed Face.*—Temperatures on the unexposed face of each wall were indicated by a set of 9 thermocouples in contact with the bare surface, by 9 thermocouples in contact with the surface but covered by standard 6 by 6-in. dry asbestos pads, and by 9 standard thermometers with the temperature sensitive portions covered by the same asbestos pads. The thermocouples and thermometers under pads were symmetrically distributed over the face of the wall in such a manner as to bring the sensitive portions opposite core spaces and above vertical mortar joints, the two places where excessive temperatures were most likely to develop due to high heat transmission or to the formation of cracks during fire exposure.

Typical locations of couples and thermometers on the unexposed face are shown in Fig. 3. All surface couples were made of No. 20 B. & S. gage calibrated iron-constantan wires, covered with asbestos insulation, and twisted and welded at the sensitive tips. They were stripped of insulation for a distance of 3½ in. from the welded tips and the bare wires were in all cases held in close contact with the unexposed surface either by the standard pads or, in the case of the bare surface temperature measurements, by a minimum of asbestos cement or gypsum applied at the insulated leads on the couples.

(*m*) *Observations During Test and Cooling Period.*—Throughout each test, observations were made regarding the character of the fire and its control, the condition of the exposed side as observed through mica windows in the furnace, and all phenomena pertinent to the performance of the wall as a fire retardant, with special reference to stability, heat insulation, and flame passage. In order to maintain conditions as uniform as practicable in the room containing the furnace, all doors and windows were kept closed during the conduct of all fire tests, regardless of season. This eliminated undesirable drafts and facilitated more reliable observations relative to steaming, smoking, etc. At the end of the fire exposure period the wall was withdrawn from the furnace and allowed to cool for 24 hours in air of laboratory. During this 24-hour cooling period reading of temperatures within the wall and at unexposed surface were made at frequent intervals until a decline from the maximum values was recorded and less frequently thereafter. All walls remained under their normal working load to permit observations of their stability during gradual cooling. Measurements of deflections and changes in dimensions as indicated by the sensitive gages were made during this entire period.

(*n*) *Loading of Wall to Failure.*—Twenty-four hours after the fire endurance test, when the wall had practically cooled to room temperature, it was gradually loaded in approximately equal increments, until the ultimate load was reached and failure occurred. Measurements of bowing and changes of dimension of the wall during the application of load were made with each load increment.

(*o*) *Photographic Records.*—Photographs were taken at both faces of each wall before and after fire exposure and after application of the ultimate load. Photographs were also taken of the unexposed face at various intervals during fire exposure.

## DISCUSSION OF RESULTS

The principal significant results obtained from a study of 12 groups of tests are presented by graphs and brief tables which bring out the influence of each factor studied upon the fire endurance period and strength of the walls over the range covered. Each figure is accompanied by brief notes giving pertinent information relative to the units and wall assembly. In the discussion which follows the influence of the several factors studied are brought out and such conclusions drawn as are justified by the results obtained.

*Basis for Evaluating Performance of Walls:*

The performance of the various test walls is compared principally on two criteria, (1) fire endurance period, and (2) stability or load-carrying ability after fire exposure.

By fire endurance period is meant the period during which the test walls sustained a working load of 80 lb. per sq. in. of gross sectional area under standard fire exposure, without transmission of flame, hot gases, and high temperatures to the unexposed side as defined by the standard fire test specifications. As only one of the walls failed under load, and none by the transmission of flame and gases hot enough to ignite ordinary combustible materials, their fire endurance periods were determined by the temperature rise on the unexposed face. The requirements of the standard are that the average temperature rise (above initial temperature of wall) of the unexposed face shall not exceed 250° F., and that the maximum rise at any point where temperature measurements are taken, shall not exceed 325° F. For all tests herein reported the fire endurance period is based upon the temperatures indicated by nine No. 20 B. & S. gage calibrated iron-constantan wire thermocouples in contact with the unexposed face under standard asbestos pads as illustrated in Fig. 3.

Figure 4, which illustrates a set of typical time-temperature curves for a wall of 3-oval core block, shows that both the maximum and minimum, as well as the average furnace temperatures, were in close agreement with the standard furnace control curve. Through this close control made possible by the aid of accurate thermocouples, all of the test walls were exposed to practically identical fire conditions so that a uniform basis for comparison of results was obtained.

The time at which the couples under pads and in contact with the unexposed face indicated an average temperature rise of 250° F. or a rise at any point of 325° F. was accurately determined to the nearest minute by means of special time-temperature graphs plotted to an enlarged scale. The *shortest time period* obtained in this manner was taken as representing the fire endurance period of each wall and is the value used in the various graphs and tables.[1] In all but a few of the tests the average temperature rise of 250° F. determined the fire endurance period rather than the maximum permissible rise of 325° F. at any individual point. In all except one case the exposure to fire was continued beyond the time the permissible temperature rise had been attained, being terminated at the next half-hour period. This made possible the comparison of wall strength at comparable fire exposure periods.

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[1] Although the particular test technique followed gives a reliable basis for comparison of the relative fire-resistant properties of the various wall assemblies, it may be of interest to point out that the fire endurance periods obtained in this investigation with $5\frac{1}{2}$ by 6-ft. walls are approximately 10 per cent lower than the periods obtained in standard tests of 10 by 11-ft. walls at Underwriters' Laboratories which were constructed of units identical in design and composition. The fire endurance periods derived in this investigation may therefore be considered as conservative criteria of the performance of standard size wall specimens.

*Excess Load Tests:*

While the Standard Specifications do not require that bearing walls subjected to fire endurance tests carry excess loads after fire exposure, ability to carry such loads, as determined from ultimate strength tests, is a very desirable property and provides direct evidence regarding the extent of the fire damage to wall assemblies. Information on the relative stability of the various walls was therefore obtained by both comparing their ultimate strengths after fire exposure and by comparing the ratios of ultimate wall strength after fire exposure to the original strength of the individual units. The latter method was of particular value and was used in most of the tests as it compensated for variations in the original strength of the units.

### GRADING OF AGGREGATE—GROUP 1

The 18 test walls in this group were built with block of the 3-oval core type (Fig. 8) of the same cement content, but of different proportions of fine to coarse aggregate. Four different aggregates representative of the following types were used: calcareous sand and gravel; siliceous sand and gravel; crushed limestone; and crushed light-weight, porous, burned shale (Haydite).

The mixed aggregate of each type was grade dfrom 0 to $\frac{3}{8}$ in.; fine aggregate in general ranged from 0 to the No. 4 sieve; coarse aggregate from No. 4 sieve to $\frac{3}{8}$ in. The different gradings desired were obtained by progressive variations in the proportion of fine to coarse material and covered the entire range of grading possible in the practical manufacture of concrete masonry units by the dry-tamped method. On the basis of the fineness modulus, which for any particular grading is an index of the average size of the particles, the grading ranged from 2.0 to 4.7. Since each type of aggregate was graded from 0 to $\frac{3}{8}$-in. size, similar values of fineness modulus represent practically equal gradings and average size of particles. It was necessary to vary the mix from 1:7.0 to 1:8.3 by volume of cement to dry rodded volumes of mixed aggregate in order to obtain as nearly as possible the same quantity of cement in the block (4.0 to 4.5 lb.).

The principal data from these tests are given in Table I.

*Fire Endurance with Different Aggregate Gradings:*

The effect of variation in grading on the fire endurance periods of the walls of the different types of aggregates are shown in Table I, there being a separate section in the table for each of the aggregates. A study of this table will show that the fire endurance period for the walls from any given aggregate increased as the proportion of the fine material in the mixture increased. For all the different aggregates, the increase in fire endurance was practically proportional to the decrease in fineness modulus. This is illustrated in the upper diagram of Fig. 5 for the calcareous aggregates. When the other data are plotted in this same way it will be found that they are represented by a series of approximately parallel lines which range above

TABLE I.—FIRE ENDURANCE AND STRENGTH FOR DIFFERENT GRADINGS OF AGGREGATE.

**Walls 8 in. thick made with 3-oval-core block.**
**Description of aggregates used are given in notes accompanying Table III.**

| Fineness Modulus | Mix by Volume | Cement Content | | Average Weight of Air-Dry Block, lb. | Fire Endurance Period, minutes | Ultimate Strength, lb. per sq. in., Gross Area | | Ratio of Wall Strength to Original Strength of Unit, per cent |
|---|---|---|---|---|---|---|---|---|
| | | Blocks per Sack | lb. per Block | | | Block Before Fire Exposure | Wall After Fire Exposure Indicated | |
| | | | | CALCAREOUS SAND AND GRAVEL AGGREGATE | | | | |
| 4.50 | 1:7.7 | 21.2 | 4.43 | 49.9 | 141 | 1860 | 450 (3 hr.) | 24.6 |
| 4.00 | 1:7.6 | 21.4 | 4.39 | 48.5 | 149 | 1100 | 205 ( " ) | 18.6 |
| 3.50 | 1:7.8 | 22.4 | 4.20 | 46.6 | 150 | 790 | 240 ( " ) | 30.4 |
| 3.00 | 1:8.2 | 23.4 | 4.01 | 44.3 | 155 | 705 | 160 ( " ) | 22.7 |
| 2.00 | 1:7.2 | 21.4 | 4.39 | 41.2 | 164 | 500 | 80 ( " ) | 16.0 |
| | | | | SILICEOUS SAND AND GRAVEL AGGREGATE "A" | | | | |
| 4.25 | 1:7.5 | 21.2 | 4.43 | 46.4 | 118 | 1810 | 480 (3 hr.) | 25 |
| 3.50 | 1:7.5 | 21.8 | 4.31 | 45.2 | 125 | 1190 | 240 ( " ) | 20 |
| 2.46 | 1:7.5 | 21.6 | 4.35 | 42.5 | 143 | 675 | 100 ( " ) | 15 |
| | | | | LIMESTONE AGGREGATE | | | | |
| 4.60 | 1:7.4 | 21.8 | 4.31 | 41.1 | 145 | 800 | 300 (3 hr.) | 37 |
| 4.40 | 1:8.3 | 21.6 | 4.35 | 46.0 | 147 | 1030 | over 240[a] ( " ) | .. |
| 3.50 | 1:7.9 | 21.7 | 4.33 | 49.0 | 158 | 1580 | 530 ( " ) | 33 |
| 2.83 | 1:7.7 | 21.7 | 4.33 | 47.2 | 161 | 1170 | 470 ( " ) | 40 |
| 2.16 | 1:7.5 | 21.6 | 4.35 | 46.8 | 172 | 1100 | 460 ( " ) | 42 |
| | | | | HAYDITE AGGREGATE | | | | |
| 4.75 | 1:7.0 | 21.4 | 4.39 | 23.8 | 146 | 480 | 145 (3½ hr.) | 30 |
| 4.25 | 1:7.0 | 21.1 | 4.45 | 25.6 | 159 | 680 | 273 ( " ) | 40 |
| 3.75 | 1:7.0 | 21.3 | 4.41 | 27.2 | 162 | 890 | 324 ( " ) | 36 |
| 3.00 | 1:7.0 | 21.2 | 4.43 | 28.2 | 175 | 930 | 333 ( " ) | 36 |
| 2.07 | 1:7.3 | 21.0 | 4.47 | 28.5 | 180 | 840 | 273 ( " ) | 32 |

[a] Ultimate strength of wall not determined.

and below the graph for the calcareous material which represents about average fire endurance periods for the group.

In this connection it is interesting to note the large variations in weight of units resulting from the different combinations of fine and coarse aggregate of each type. These are shown by the upper set of curves in Fig. 6. Units of the same aggregate showed a range in weight of from 19 to 25 per cent (based on the minimum value). The wide differences in the shape of these curves are largely due to the void characteristics of the aggregates which in turn are affected by the shape and surface characteristics, particularly the angularity and roughness of the aggregate particles themselves. In view of these variable effects of grading upon the weight of the units and upon the denseness and internal structure of the concrete, it is rather surprising that the influence of grading upon the fire endurance period can be expressed by such a simple straight-line relationship as that shown in Fig. 5. It is believed that these results are due to the fact that an increase in the proportion of fine aggregate is always accompanied by a corresponding increase in the number of small air voids between the aggregate particles, and may, under certain conditions, also be accompanied by a decrease in the size of the comparatively large air voids between the coarse aggregate particles. On this basis it appears that the transmission of heat through tamped concrete

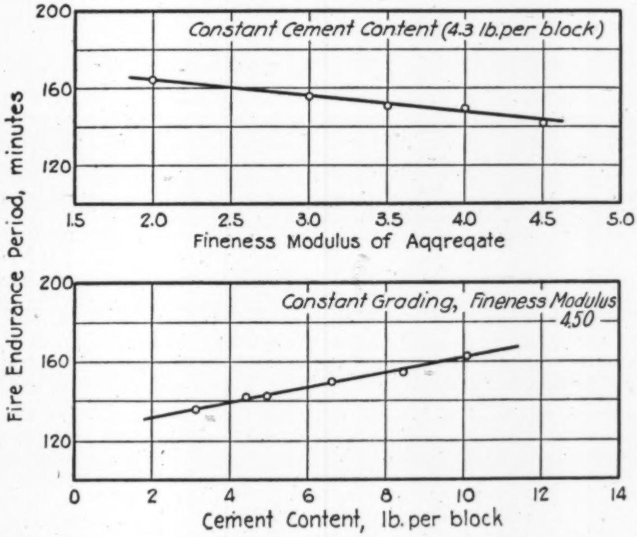

FIG. 5.—Effect of Grading of Aggregate and Cement Content on Fire Endurance Period of 8-in. Walls of 8 by 8 by 16-in. 3-Oval-Core Block Units Made with Calcareous Sand and Gravel.

decreased as the number of small voids increased and the size of the large voids decreased. This is in harmony with similar well-known principles governing the properties of various insulating materials at both high and low temperatures.

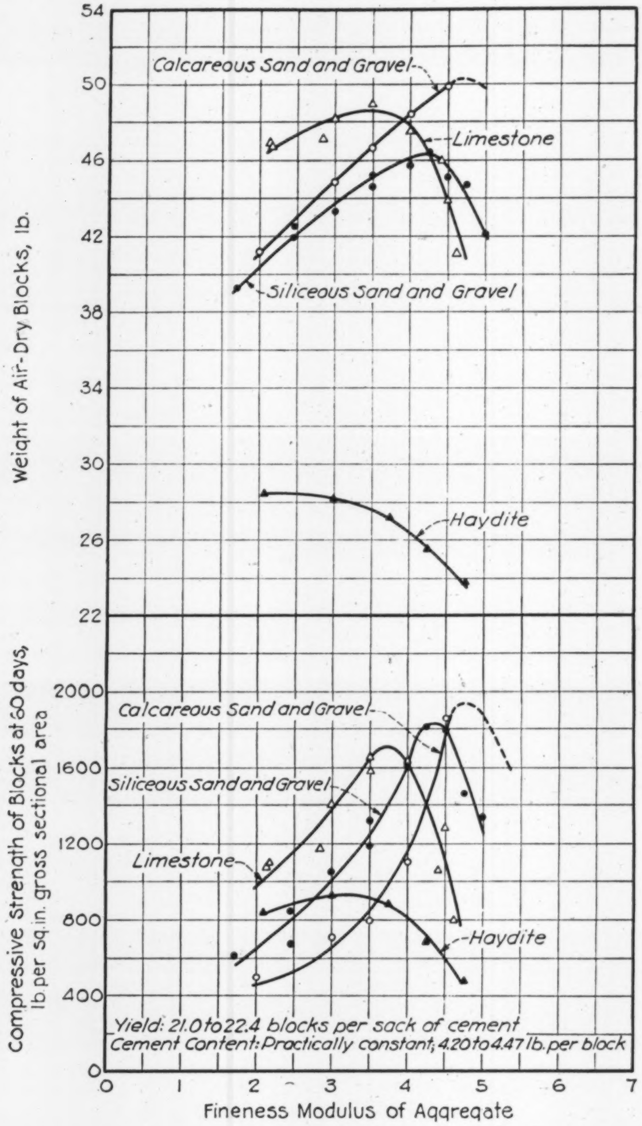

FIG. 6.—Effect of Grading of Aggregate on Weight and Compressive Strength of Air-Dry 8 by 8 by 16-in. 3-Oval-Core Blocks.

This effect of the size and distribution of the voids is well illustrated by the tests of block made with the crushed limestone and the siliceous sand and gravel. It will be noted from the weight graphs of the blocks for these materials in Fig. 6 that different proportions of of fine and coarse aggregate above and below the proportion giving maximum weight, produced blocks of equal weight and of the same apparent denseness. Therefore with blocks of the same weight and cement content but of different grading, the total volume of voids consists either of a large number of small voids in the case of the fine grading or of a small number of comparatively large voids in the case of coarse grading. The fire endurance periods obtained under these conditions (Table I) show that, from the standpoint of heat transmission, the performance of tamped concrete made from the same weight and volume of aggregate and with the same quantity of cement is improved by distributing the material so as to give the largest number of small voids. The coarser gradings which produce "popcorn concrete" should be avoided.

*Strength of Walls and Individual Units:*

The lower set of curves in Fig. 6, based on the data in Table I, shows the large effect upon the compressive strength of the blocks due to variations in grading. Based on minimum values, the variations in strength of blocks from the same aggregate range from 200 to 300 per cent. Compression tests to determine the strength of walls not exposed to fire for comparison with strength of comparable walls after fire exposure showed similar variations in wall strength and indicated that the load-carrying capacity of the walls made from each type of aggregate was proportional to the original strength of the units. These tests also showed that the ratio between the strength of wall and original strength of the unit was practically constant for a given type of aggregate. This general relationship between wall and unit strength was established not only from the tests in this group but also from numerous other tests made in this investigation, particularly those in group 2 in which the strength of the units was varied between even wider limits by changing the cement content. The fact that the wall strengths before and after fire exposure bears a direct relationship to the original strength of the units is well illustrated by the curves in Figs. 14 and 15 which are based on the combined data originating from the various groups of tests and which will be more completely discussed later.[1]

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[1] See p. 652.

### CEMENT CONTENT—GROUP 2

In this group 14 walls were built with units of the same design (3 oval core block, Fig. 8) and of a wide range in cement content. Three types of aggregate, identical with the calcareous sand and gravel, the siliceous sand and gravel and the Haydite of group 1 were used. The grading of each type was held constant at about the fineness modulus found to yield maximum strength of unit in the tests of group 1 (see strength curves of Fig. 6). These particular gradings (fineness moduli 4.50, 4.25 and 3.25 respectively for the three types of aggregates) provided units of standard strength[1] with exceedingly lean mixes, and units of very great strength and denseness with rich mixes. As in group 1 fine aggregate in general ranged

TABLE II.—FIRE ENDURANCE AND STRENGTH FOR DIFFERENT CEMENT CONTENTS.

Walls 8 in. thick made with 3-oval-core blocks.
Descriptions of aggregates used are given in notes accompanying Table III.

| Fineness Modulus | Mix by Volume | Cement Content | | Average Weight of Air-Dry Block, lb. | Fire Endurance Period, minutes | Ultimate Strength, lb. per sq. in. Gross Area | | Ratio of Wall Strength to Original Strength of Unit, per cent |
|---|---|---|---|---|---|---|---|---|
| | | Block per Sack | lb. per Block | | | Block Before Fire Exposure | Wall After Fire Exposure Indicated | |
| CALCAREOUS SAND AND GRAVEL AGGREGATE | | | | | | | | |
| 4.50 | 1:14.0 | 40.0 | 2.35 | 46.4 | 133 | 920 | 270 (3 hr.) | 29 |
| 4.50 | 1:11.0 | 30.0 | 3.13 | 48.5 | 135 | 1160 | 310 ( " ) | 27 |
| 4.50 | 1:7.0 | 19.0 | 4.95 | 51.1 | 142 | 2440 | 560 ( " ) | 23 |
| 4.50 | 1:5.0 | 14.3 | 6.58 | 51.6 | 149 | 2300 | 550 ( " ) | 24 |
| 4.50 | 1:4.0 | 11.1 | 8.45 | 52.4 | 154 | 3350 | 615 ( " ) | 18 |
| 4.50 | 1:3.0 | 9.3 | 10.10 | 52.4 | 162 | 3550 | 750 ( " ) | 21 |
| SILICEOUS SAND AND GRAVEL AGGREGATE "A" | | | | | | | | |
| 4.25 | 1:14.0 | 38.5 | 2.44 | 44.5 | [a] | 725 | [a] | [a] |
| 4.25 | 1:7.5 | 21.2 | 4.42 | 46.4 | 118 | 1810 | 480 (3 hr.) | 26 |
| 4.25 | 1:3.0 | 9.3 | 10.10 | 49.6 | 137 | 4075 | 855 ( " ) | 21 |
| HAYDITE AGGREGATE | | | | | | | | |
| 3.25 | 1:10.0 | 29.1 | 3.23 | 26.8 | 160 | 750 | 256 (3½ hr.) | 34 |
| 3.25 | 1:8.0 | 23.6 | 3.98 | 27.8 | 174 | 1000 | 445 ( " ) | 44 |
| 3.25 | 1:7.0 | 21.1 | 4.45 | 28.2 | 178 | 1120 | 358 ( " ) | 32 |
| 3.25 | 1:5.0 | 15.3 | 6.15 | 30.0 | 194 | 1575 | 580 ( " ) | 37 |
| 3.25 | 1:3.0 | 9.8 | 9.60 | 32.8 | 221 | 2100 | 770 (4 " ) | 37 |

[a] This wall failed at 93 minutes of fire exposure under the working load of 80 lb. per sq. in. of gross area and was withdrawn from the furnace at 94 minutes. Readings of temperatures on the unexposed surface which were continued indicated that an average rise of 250° F. was attained at 114 minutes and the maximum rise of 325° F. at 122 minutes. Had the exposure to fire been continued for about 20 minutes longer, it is estimatedthat the average temperature rise of 250° F. would have been attained about 5 minutes earlier or at 109 minutes.

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[1] An average strength not less than 700 lb. per sq. in. of gross sectional area as laid in the wall is required by the standards for concrete block and tile of the American Concrete Institute (Serial Designation P-1 A-29 S) and the Underwriters' Laboratories. This strength is also prescribed in Section 9, page 9, of the "Recommended Minimum Requirements for Masonry Wall Construction," June 26, 1924, prepared by the Building Code Committee of the U. S. Department of Commerce

from 0 to No. 4 sieve and coarse aggregate from the No. 4 sieve to $\frac{3}{8}$ in.

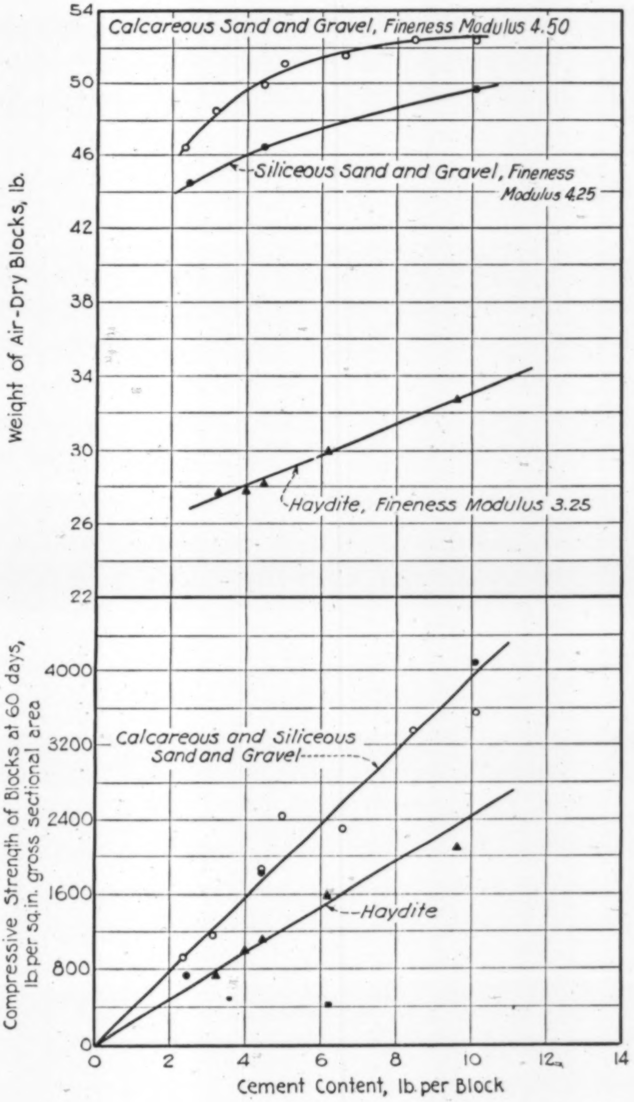


FIG. 7.—Effect of Cement Content on the Weight and Compressive Strength of Air-Dry 8 by 8 by 16-in. 3-Oval-Core Blocks.

The Haydite units were made from mixes ranging from 1:10 to 1:3 and contained from 3.23 to 9.62 lb. of cement per unit. The sand and gravel units were made from mixes ranging from 1:14 to 1:3 and contained from 2.35 to 10.10 lb. of cement per unit. All mixes were by volume of cement to dry rodded volumes of mixed aggregate.

The principal data from this group of tests are shown in Table II.

*Fire Endurance Periods for Different Cement Contents:*

The lower curve in Fig. 5, which represents the results of fire tests of walls of the calcareous aggregate in this group, shows that the fire endurance period steadily increased as the cement content increased and that the relation between fire endurance period and the weight of cement per block is represented by a straight line. When the respective fire endurance periods for the other aggregates in Table II are similarly plotted straight-line relations also result. This proportional increase in fire endurance period with increase in cement was to have been expected since a large part of the resistance to the transmission of heat is due to the dehydration of the cement which occurs when concrete is exposed to high temperatures. The more hydrated cement available for dehydration, the greater is the potential fire endurance.

In the case of the Haydite walls, the fire endurance period increased at a substantially greater rate in proportion to cement content than in the case of the walls of sand and gravel aggregate. It is believed that this difference in behavior is due primarily to the influence of the larger quantity of water absorbed and retained by the porous Haydite particles during molding and moist curing as compared with that retained by the more dense sand and gravel particles. As the cement paste becomes richer, and therefore more impermeable, more water remains sealed in the vesicular structure of the Haydite concrete. The vaporization of this water absorbs heat and therefore prolongs the fire endurance period beyond the time that would logically result from the dehydration of the cement alone.

The fact that the fire endurance period remained proportional to the cement content, in spite of the accompanying variations in the structural characteristics of the blocks is of interest in showing the greater influence of the chemically active ingredient as compared to variations in physical structure. These physical variations are plainly revealed by the upper set of curves in Fig. 7 which show the general increase in the weight of the units (with the consequent increase in the denseness of the concrete) resulting from the substitution of cement for air in the spaces between the aggregate particles.

*Relation of Strength of Wall to Original Strength of Unit:*

The relation of strength of units to cement content is given in the lower set of graphs in Fig. 7 which show that the strength of units made with each type of aggregate was for all practical purposes directly proportional to the cement content. The relations for the units of the two sand and gravel aggregates are represented by the same graph which illustrates the very great range in the strength of the units (725 to 4075 lb. per sq. in.) used in laying up walls of this group. As in group 1 the strength of the walls both before and after fire exposure, within the limits of these tests, was closely proportional

TABLE III.—FIRE ENDURANCE AS AFFECTED BY CHARACTER OF AGGREGATES.

Walls 8 in. thick made with 3-oval-core block.

Description of aggregates:

*Siliceous Sand and Gravel* (B): A highly siliceous aggregate (85 per cent silica in form of quartz) containing very little calcareous material. Particles range from regular to irregular in shape but generally with well-rounded edges and hard smooth surfaces.

*Siliceous Sand and Gravel* (A): A somewhat more siliceous aggregate than aggregate (B) (95 per cent silica in form of flint and chert). Particles very irregular with both sharp and slightly rounded edges and both polished and pitted surfaces.

*Calcareous Sand and Gravel:* A typical calcareous aggregate containing about 40 per cent calcium carbonate, 30 per cent magnesium carbonate, and less than 15 per cent quartz. Particles range from regular to irregular in shape but generally with well-rounded edges. Surfaces varied from smooth, slippery texture to a rough porous pitted type with neither type predominating.

*Crushed Limestone:* Typical of crushed calcareous stone of dolomitic variety containing roughly 50 per cent of calcium carbonate, 30 per cent magnesium carbonate, and with less than 5 per cent quartz. Particles very irregular in shape with slightly rounded sharp edges. Surfaces were somewhat slippery to the touch and slightly porous.

*Crushed Common Brick:* Typical of the fine and coarse low fusing porous burned clay aggregate produced by crushing common brick. Particles very irregular in shape with sharp rough edges and rough pitted surfaces.

*Crushed Fire Brick:* Typical of the fine and coarse highly refractory porous burned clay aggregate produced by crushing broken pieces of fire brick and other shapes of similar refractory material. Particles very irregular in shape with very rough edges and deeply pitted exceedingly rough surfaces.

*Haydite:* A light, porous, aggregate material prepared by burning shale to incipient fusion and crushing and grading the resulting clinker. Particles irregular in shape with slightly rounded sharp edges and slightly rough porous and pitted surfaces. Unless otherwise noted the Haydite aggregate used in this and other groups of tests came from the same source.

*Air-Cooled Blast Furnace Slag:* Typical of the slag discharged from blast furnace and brought from the molten to solid form by cooling in air.

*Soft Coal Cinders:* Typical of the porous clinkerous ash resulting from the high temperature combustion of soft coal in the modern power plant. It contained about 12 per cent of combustible material principally in the form of coke and to a minor extent as soft coal. Particles irregular in shape with both sharp and well-rounded edges and both glossy and rough pitted porous surfaces.

*Coke Breeze:* This material consists of screenings ranging from dust to $\frac{3}{8}$ in. of the highly porous product commonly used as fuel known as coke. It consists almost entirely of carbon (ash 12 to 14 per cent) and is the product remaining after the hydrocarbon gases have been distilled from soft coal in the manufacture of gas. Particles irregular in shape with slightly rounded sharp edges and rough deeply pitted exceedingly porous surfaces.

| Aggregate | | | Mix by Volume | Cement Content | | Average Air-Dry Weight, lb. | | Fire Endurance Period, minutes | |
|---|---|---|---|---|---|---|---|---|---|
| Type | Fineness Modulus | Unit Weight, lb. per cu. ft.[a] | | Blocks per Sack | lb. per Block | Per Block | Per sq. ft. of Wall | Wall | Per lb. per sq. ft. of Wall |
| Coke Breeze | 3.50 | 49 | 1:7.5 | 21.2 | 4.44 | 23.5 | 29.0 | 110 | 3.79 |
| Siliceous Gravel A | 3.50 | 115 | 1:7.5 | 21.8 | 4.30 | 45.2 | 53.2 | 124 | 2.33 |
| Siliceous Gravel B | 3.70 | 113 | 1:8.3 | 22.5 | 4.18 | 46.7 | 54.9 | 129 | 2.35 |
| Crushed Fire Brick | 3.50 | 91 | 1:7.5 | 21.9 | 4.29 | 36.8 | 43.8 | 145 | 3.21 |
| Calcareous Gravel | 3.50 | 120 | 1:7.8 | 22.4 | 4.20 | 46.6 | 54.8 | 150 | 2.74 |
| Crushed Limestone | 3.50 | 120 | 1:7.8 | 21.7 | 4.33 | 49.0 | 57.4 | 158 | 2.75 |
| Soft Coal Cinders | 3.25 | 79 | 1:7.0 | 20.6 | 4.56 | 33.9 | 40.6 | 163 | 4.01 |
| Haydite | 3.50 | 67 | 1:7.0 | 21.2 | 4.43 | 27.6 | 33.5 | 168 | 5.01 |
| Crushed Common Brick | 3.50 | 77 | 1:7.5 | 21.6 | 4.36 | 32.3 | 38.8 | 175 | 4.51 |
| Air-Cooled Slag | 3.50 | 102 | 1:7.5 | 22.0 | 4.25 | 41.1 | 48.7 | 193 | 3.96 |

[a] Based on dry rodded 0 to $\frac{3}{8}$-in. aggregate graded to fineness modulus indicated.

to the original strength of the units. This may be seen from Fig. 14 where the combined data of groups 1 and 2 are plotted.

### TYPE OF AGGREGATE—GROUP 3

Table III gives the details and performance of a series of 10 walls built up of units of identical design but which differed principally in the type of aggregate and to a minor extent in grading and cement content. These 10 walls were all 8-in. thick laid up with units of 3-oval core block design (Fig. 8) and with workmanship designated as "good" in Fig. 13 (*a*). The units varied widely in porosity, weight, strength, and mineral composition. Each type of aggregate was graded from 0 to $\frac{3}{8}$ in. Fine aggregate, in general, ranged from 0 to the No. 4 sieve; coarse aggregate from the No. 4 sieve to $\frac{3}{8}$ in. In general, the different aggregates were graded to have a constant fineness modulus of 3.50. The proportion of cement to aggregate was varied from 1:7.0 to 1:8.3 in order to obtain as nearly as possible the same quantity of cement per block (4.18 to 4.56 lb.).

For convenience of study, the test walls of the different aggregates in Table III are arranged in the order of ascending values of fire endurance periods. The range of 83 minutes between the highest and lowest values shows the influence of type of aggregate on the fire endurance period. This range represents a variation of approximately 27 per cent above and below the average period of 152 minutes. The difference in fire endurance period is due to differences in the physical properties and mineral composition of the individual aggregates, as well as to the difference in the physical structure of the resulting units. When the performance of the aggregates is compared by their fire endurance periods per pound of material per square foot of wall (last column, Table III), the lighter materials are seen to be, in general, more effective than the heavier material.

The limits of this paper do not permit of an adequate discussion of the relative merits of the different aggregates on the basis of economy for a given strength and fire endurance period. As pointed out elsewhere, however, it can be shown that by proper selection of type of unit, cement content, grading of aggregate and wall thickness any particular requirements can be met with any of the materials tested.

### DESIGN OF UNIT—GROUP 4

The influence of design of unit was studied from a series of tests of 8 walls, using the six designs detailed in Figs. 8 and 9. All units were made of dry-tamped concrete identical as to cement content (per pound of unit), type and grading of aggregate, consistency and curing.

Five of the walls were 8 in. thick, two were 12 in., and one was 4 in.

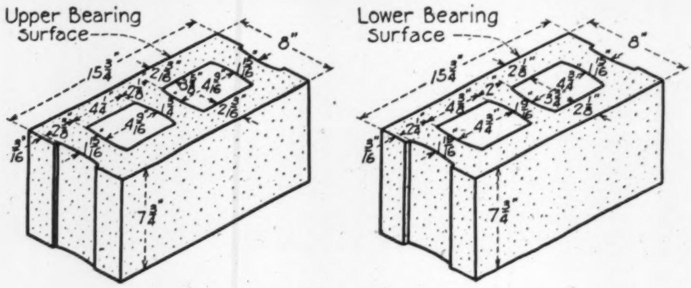
Calcareous sand and gravel has been principally used in this group, because it is a natural aggregate, fairly uniform as to physical properties, and of mineral composition representative of calcareous gravels in common use and having average fire endurance and heat transmission properties. The fine aggregate was graded from 0 to the No. 4 sieve; coarse aggregate from the No. 4 sieve to $\frac{3}{8}$ in. The fineness modulus of the mixture was held constant at 4.0. The mix was approximately 1:8 by volume of cement to dry rodded volume of mixed aggregate.

The sectional diagrams, numbered from 1 to 8 in Fig. 10, illustrate the 8 different types of walls that were constructed from the six block and tile designs. In the diagram at the left of Fig. 10, the relation between the fire endurance period and the average weight per sq. ft. of wall is shown. In this diagram, each one of the points numbered from 1 to 8 represents a single test of a panel of one of the types illustrated at the right. The points lettered 8a, 7b, etc. were obtained by computing what would be the weight and fire endurance of the portions of the walls represented by the exposed face shell or this and the intermediate parallel webs of the wall unit, using the temperatures observed within the block as a basis for the fire endurance period.

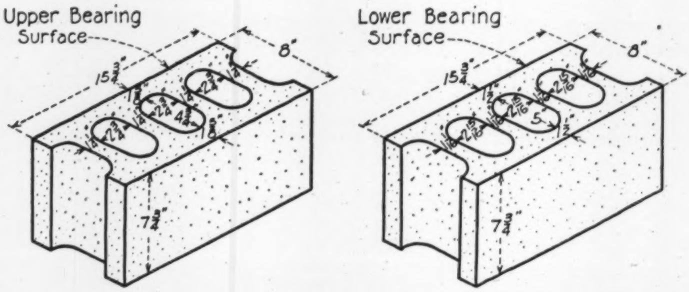
The extreme regularity of the curve representing the relation between fire endurance period and the weight of the wall will be seen to be rather remarkable when the wide variation in weight per square foot and the differences in design of units are considered. The close agreement of the points representing the fractional portions of the walls makes this regularity in relationship still more striking.

The weight per square foot of each wall is based on the air-dry weight of units and mortar used, divided by the total area of the wall. Similarly, the weight per square foot of the fractional portions of the walls nearest the fire side, and designated as "A" and "B" in types 3, 4, 6, 7 and 8, was based on the calculated air-dry weight of the fractional units and mortar used in the particular portion considered.

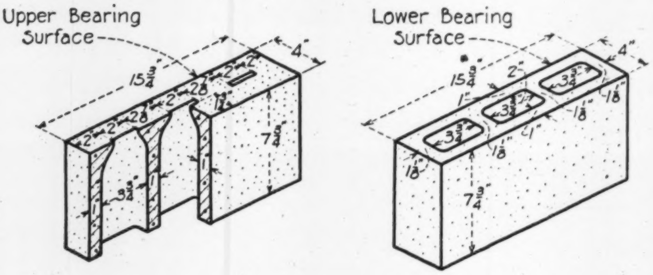
It will be seen from the regularity of the relationship that exists between the weight of the wall and the fire endurance period that some of the other factors, which would normally be considered of great importance, are of less concern. For example, differences in the type of unit or thicknesses of the face shells affect the fire endurance period only as they affect the weight of the wall. This is well illustrated by the comparison of tests of the 8-in. wall, types 3, 4, 5 and 6, in which the weights and fire endurance periods are approximately the same, regardless of the wide variation in type of unit. A comparison of types 1 and 2 shows that the total thickness of the face shells of the units does not furnish a reliable basis of judging the probable fire endurance period. In these tests, the fire endurance period with type 2 was twice that of type 1, although the combined thickness of the face shells was 2 in. in both designs. Similarly, the tests of types 7 and 8 show the fire endurance period to be independent of the num-


Upper Bearing Surface
Lower Bearing Surface
15¾"
8"
7¾"

8 × 8 × 16-in. 2-Core Block Unit


Upper Bearing Surface
Lower Bearing Surface
15¾"
8"
7¾"

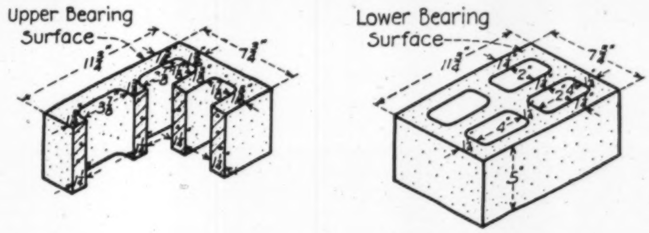
8 × 8 × 16-in. 3-Oval-Core Block Unit


Upper Bearing Surface
Lower Bearing Surface
15¾"
4"
7¾"
3¾"

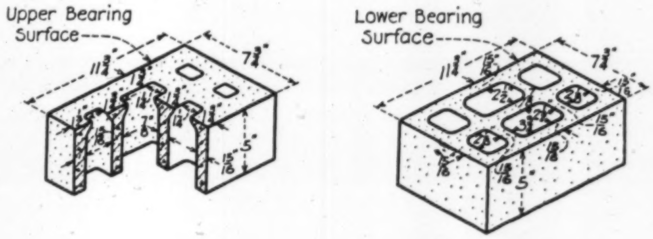
4 × 8 × 16-in. 3-Core Partition "Tile" Unit

FIG. 8.—Details of Various Designs of Concrete Masonry Units Used in Fire Endurance Tests.

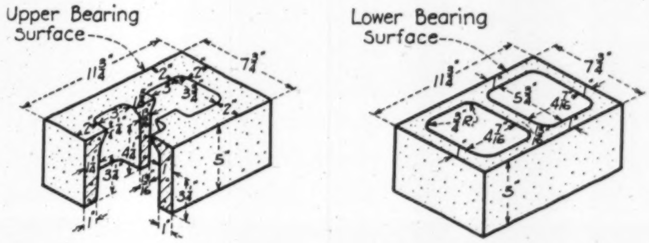
ber and shape of cells into which the wall thickness is divided, for with approximately the same weight per square foot of wall these two types gave almost identical fire endurance periods.


Upper Bearing Surface
11¾"
7¾"
Lower Bearing Surface
11¾"
7¾"
5"

5 × 8 × 12-in. 4-Core "Tile" Unit


Upper Bearing Surface
11¾"
7¾"
5"
Lower Bearing Surface
11¾"
7¾"
5"

5 × 8 × 12-in. 6-Core "Tile" Unit


Upper Bearing Surface
11¾"
7¾"
5"
1"
Lower Bearing Surface
11¾"
7¾"
5"

5 × 8 × 12-in. 2-Core "Tile" Unit

FIG. 9.—Details of Various Designs of Concrete Masonry Units Used in Fire Endurance Tests.

Preliminary information from tests of other aggregates along the lines of this group has shown curves of a similar character but ranging to the right and left of that shown in Fig. 10. These data are not only of value in confirming the general relationship between weight of wall and fire endurance period but when completed will also provide a rational basis for design of wall to give any desired performance.

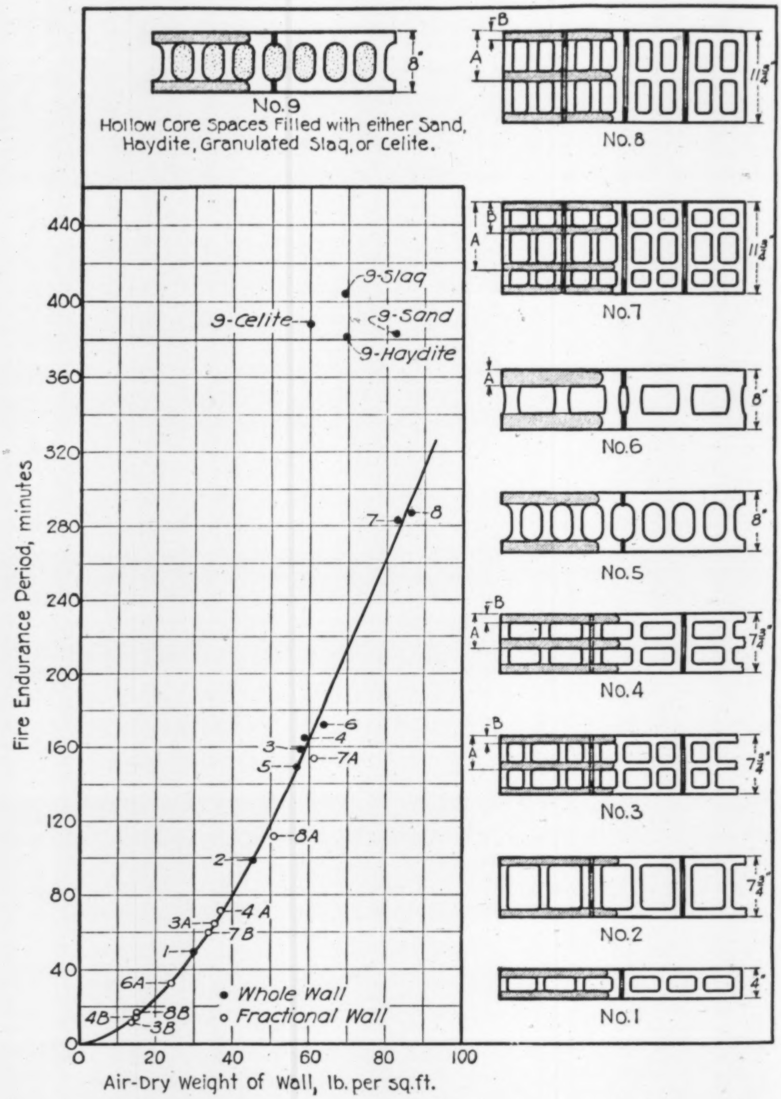


FIG. 10.—Relation Between Fire Endurance Period and Weight per square foot of Wall.

***Walls:*** **4, 8, and 12 in. thick made with different designs of units.**

*Aggregate:* 0 to $\frac{3}{8}$ in. calcareous sand and gravel of constant fineness modulus — 4.0.

*Mix:* Practically constant, 1:8 by dry rodded volume.

*Mortar:* 1:3 Portland cement plus 15 per cent hydrated lime, applied at bearing and end surfaces of units as indicated by shaded areas.

The numbers opposite the plotted points refer to the types of wall tested, sections of which are shown at right and top of diagram.

It is apparent from the foregoing discussion that from the standpoint of the fire endurance period the effectiveness of a given character of concrete cannot be improved by varying the designof unit without also changing the weight per square foot of wall. It will be clear, however, from the shape of the curve in Fig. 10 that for any additions to weight through varying the design of units, the effectiveness per pound added increases markedly with an increase in the weight per square foot of wall. It is interesting to note that, in general, an increase of 50 per cent in this weight doubled the fire endurance period throughout the range covered by the curve.

### FILLING HOLLOW WALLS—GROUP 5

The five walls in this group were identical in every respect except that after the usual seasoning period the hollow core spaces of four of the walls were filled with one of four materials: calcareous sand, crushed Haydite, granulated blast-furnace slag, or diatomaceous earth (Celite). The filling material was in each case oven dried. The fifth wall was left unfilled for comparison with the filled walls.

The walls were laid up with units of 3-oval core design (see Fig. 8) and with workmanship designated as "good" in Fig. 13 (*a*). Units of this design were particularly well adapted for this group of tests as the series of continuous vertical hollow spaces extending from the bottom to the top of the finished walls, as shown by section *C-C* of Fig. 13, were easily filled with the various materials. The units were made from calcareous sand and gravel of the same grading and mix as used in group 4.

With the exception of the Celite, the materials used to fill the various walls were graded from 0 to the No. 4 sieve with a fineness modulus of 2.60 for the slag, 2.27 for the Haydite and 2.68 for the sand. The weight of the Celite, slag, Haydite, and sand on a dry loose basis was respectively 13, 49, 61 and 106 lb. per cu. ft. and on a dry rodded basis, 13.8, 57, 68, and 114 lb. per cu. ft. The weights required to fill the hollow spaces of the walls were: Celite, 2.7, slag 11.8, Haydite 12.9 and sand 25.7 lb. per sq. ft. of wall.

Sectional diagrams No. 9 and No. 5 in Fig. 10 illustrate the essential differences between the filled and unfilled walls. Comparison of the group of plotted points marked 9—"Celite," "slag," "Haydite," and "sand" with plotted point 5 in the diagram at the left of Fig. 10 reveals the very great increase in fire endurance period that resulted from filling the hollow wall spaces with various materials. Regardless of the type of material used, the fire endurance period of the filled walls (over 380 minutes) was at least 2½ times that of the unfilled wall (150 minutes).

Contrary to what might be expected, the fire endurance periods of the various filled walls showed a variation of less than 4 per cent above and below the average of about 390 minutes for the group. On the basis of weight per square foot of wall, the lighter materials show an appreciable advantage over the wall filled with sand. In this connection, it is very interesting to note that the fire endurance period of the wall filled with sand was greater by about 100 minutes or 35 per cent than that for hollow walls 12 in. thick of types 7 or 8 which had the same weight per square foot and which were made from the same type of concrete.

Each of the filled walls was exposed to fire for 6½ hours and tested for strength after cooling to room temperature. The results showed an average ratio of strength of wall to original strength of unit for the group of 20 per cent as compared to 28 per cent for the unfilled wall after 3 hours of fire exposure. This indicates that filled walls made of units having a strength of 700 lb. per sq. in. would be able to carry the normal working load of 80 lb. per sq. in. of gross sectional wall area during and after 6½ hours of severe fire exposure with a margin of safety of at least 50 per cent.

### EFFECT OF PLASTER—GROUP 6

In this group, a number of 8-in. walls were tested to study the effect of plaster on the fire endurance and stability. Six walls were built up of 3-oval-core blocks using two aggregates of widely different types. One wall of each aggregate was plastered on the fire face, one on the unexposed face and one on both faces. The performance of these walls was compared to that of unplastered walls of otherwise identical make-up from group 2. The two aggregates used were the calcareous sand and gravel and Haydite. To eliminate in so far as possible the direct and indirect effects of cement content, the units selected for these tests were of the leanest mixes shown in group 2.

The Haydite aggregate was graded to a fineness modulus of 3.25 and the mix was 1:10. The calcareous aggregate was graded to a fineness modulus of 4.50 and the mix was 1:14. Throughout the group the walls were laid up with 1:3 portland-cement mortar (plus 15 per cent hydrated lime by volume of cement) with workmanship designated as "good" in Fig. 13(*a*).

Gypsum plaster was used, applied in three coats with a combined thickness of not less than ⅜ in. nor more than ½ in. The first or scratch coat consisted of "bondcrete," a wood pulp gypsum plaster applied without sand to a thickness of about $\frac{1}{16}$ in. The second or brown coat consisted of a mixture of 1 part of hair-fibered gypsum plaster to 4 parts by weight of dry lake sand applied to a thickness of about $\frac{3}{8}$ in. The third or finish coat consisted largely of lime putty with the addition of gypsum gaging and fibered plaster and was applied to a thickness of about $\frac{1}{16}$ in. The weight per square foot of the dry set materials in the three coats ranged as follows: scratch coat 0.50 to 0.55 lb.; brown coat 3.4 to 3.5 lb.; finish coat 0.50 to 0.55 lb. On this basis the total weight of the three coats ranged from 4.4 to 4.6 lb. and averaged about 4.50 lb. per sq. ft. The walls were seasoned from 45 to 60 days before plastering and from 70 to 85 days after plastering in freely circulating air at the temperature and humidity of a heated building.

*Effect of Plaster on Fire Endurance Period:*

The effect of plaster finish was to retard the transmission of heat and increase substantially the fire endurance period. The fire endurance periods of the plastered and unplastered walls are shown in Table IV, from which it will be seen that the fire endurance periods were increased by about 30 per cent with $\frac{1}{2}$ in. of plaster on either face and about 60 per cent for $\frac{1}{2}$ in. of plaster on both faces.

TABLE IV.—EFFECT OF PLASTER ON FIRE ENDURANCE AND STRENGTH OF 8-IN. WALLS MADE WITH 3-OVAL-CORE BLOCK.

| Plastered Face | Fire Endurance Period of Wall, minutes | | Percentage Increase on Unplastered Wall | | Ratio of Wall Strength, After Fire Exposure Indicated, to Original Strength of Unit, per cent | |
|---|---|---|---|---|---|---|
| | Elgin Aggregate | Haydite Aggregate | Elgin Aggregate | Haydite Aggregate | Elgin Aggregate | Haydite Aggregate |
| None........... | 133 | 160 | .. | .. | 28 (3 hr.) | 36 (3½ hr.) |
| Fire Face........ | 178 | 197 | 34 | 23 | 32 (3½ hr.) | 43 (3½ hr.) |
| Unexposed Face.. | 172 | 212 | 29 | 32 | 29 (3½ hr.) | 43 (3½ hr.) |
| Both Faces...... | 209 | 267 | 57 | 67 | 39 (4 hr.) | 43 (4½ hr.) |

Judging from the tests in group 4 which showed that the effectiveness of the concrete increased with the weight per square foot of wall, it appears that the same quantity of plaster would be more effective when applied to bare walls having initially higher fire endurance periods, either through composition or design of unit, than when used on walls of exceedingly lean units as used in this group.

With walls constructed of the same type of units it is obvious that the increase in the fire endurance period will be greater or less depending on whether the plaster finish is thicker or thinner than the $\frac{1}{2}$-in. thickness applied in these tests.

With the exception of the thin finish coat of lime putty, the plaster on the fire side remained in position during the entire fire exposure period (3½ to 4½ hours) and subsequent cooling. This is significant in view of wide differences in the surface characteristics and relative expansion and contraction of the concrete units to which the plaster was bonded. The three coats of plaster on the unexposed face remained in place in all cases.

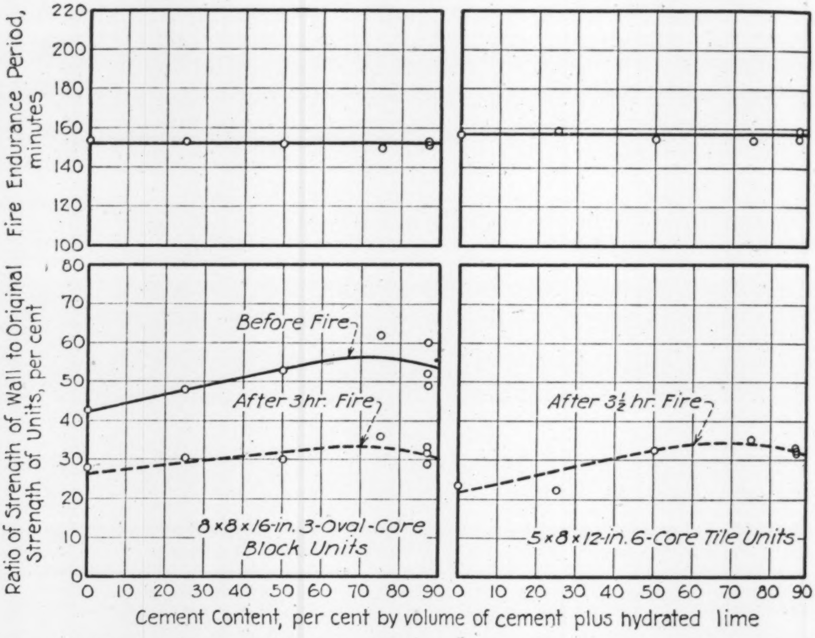


FIG. 11.—Effect of Type of Mortar on Fire Endurance Periods and Ratio of Strength of Wall to Original Strength of Units.

**Block and tile units made from 1:8 mix by dry, rodded volume of Elgin sand and gravel aggregate of fineness modulus 4.1.**

**Walls laid with mortars consisting of 1 volume of cementitious material (portland cement plus hydrated lime) to 3 volumes of dry, rodded Elgin sand graded 0 to No. 14 sieve. These 1:3 mixes correspond approximately to 1:4 mixes proportioned in field with damp, loose sand. Desired volumetric mortar proportions maintained by using equivalent weight proportions of dry materials considering weight of sand to be 110 lb., cement 94 lb., and lime 42 lb. per cu. ft.**

**Block units bedded in mortar at face shells only as shown in Fig. 13 (*a*).**

**Tile units bedded in mortar at face shells and central web as shown in Fig. 12 (*b*).**

**Walls exposed to fire allowed to cool to room temperature for 24 hours before loading to failure.**

**Age of walls when tested, 2 months.**

*Effect of Plaster on Wall Strength:*

Since the period during which the bare and plastered walls of calcareous and Haydite aggregate were exposed to fire varied from 3 to 4½ hours, the effect of plaster on strength after fire exposure can be determined only in a general way from a comparison of the ratios of wall strength to original strength of unit in Table IV. These indicate that the strength of the walls when plastered on both faces was substantially greater after 4 to 4½ hours of fire exposure than that of the bare walls after 3 to 3½ hours fire exposure. Walls made of the calcareous aggregate and plastered on either the exposed or unexposed faces were somewhat stronger after 3½ hours of fire exposure than the bare wall after 3 hours. Walls made with Haydite and plastered on either the exposed or unexposed faces were considerably stronger after 3½ hours of fire exposure than the bare wall exposed for the same period. It appears therefore that with both types of aggregate, the application of plaster to one or both faces of the wall resulted in an appreciable improvement in strength after the same or somewhat longer periods of fire exposure.

### TYPE OF MORTAR—GROUP 7

*Details of Walls:*

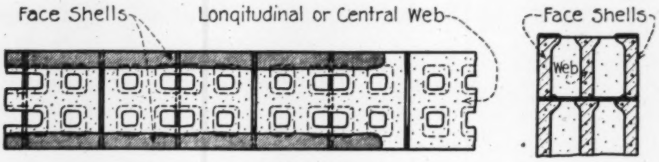
This group comprised tests on 14 walls of 3-oval core block (Fig. 8) and 7 walls of 6-core tile units (Fig. 9) laid up with 5 different types of nominally 1:3 mortar, ranging from straight lime mortar to portland-cement mortar plus 15 per cent hydrated lime.

Throughout the group, walls of each design of unit were laid up with workmanship designated as "good" in Figs. 13(*a*) and 12(*b*). Both block and tile units were made from calcareous sand and gravel of identical grading and mix as used in group 1.

Each of the five mortars used consisted of 3 volumes of dry rodded Elgin sand (0 to No. 14 sieve, fineness modulus 1.90 to 2.00) to 1 volume of cementitious material. The percentages of portland cement used were: 0, 25, 50, 75, and 87, with corresponding lime contents of 100, 75, 50, 25, and 13 per cent all by *volume*. As explained in the notes accompanying Fig. 11 which shows the results of these tests, these nominally 1:3 volumetric proportions of mortar materials on a dry rodded basis correspond approximately to a 1:4 field mixture using damp and loose sand. Chemical analysis of the 0 to No. 14 sand used in mixing the mortars showed that about half of the particles were calcareous and half siliceous in character. The walls were seasoned in freely circulating air at the temperature and humidity of a heated building for 60 days before testing.

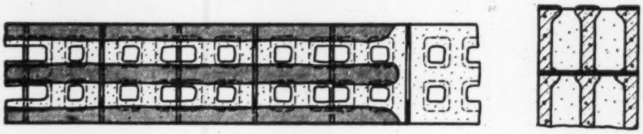
*Influence of Type of Mortar on Fire Endurance:*

The upper set of curves in Fig. 11 show that the fire endurance period was practically unaffected by wide variations in the type of mortar used in laying either the large 8 by 8 by 16-in. block units or the smaller 5 by 8 by 12-in. tile units. This appears reasonable since the composition of the wall assemblies as a whole was but slightly



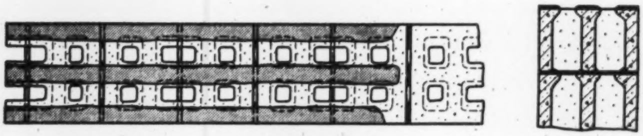
(a) FAIR

Horizontal Joints: Bedded at Face Shells only
Vertical Joints: Two Strips of Mortar with One Air Space



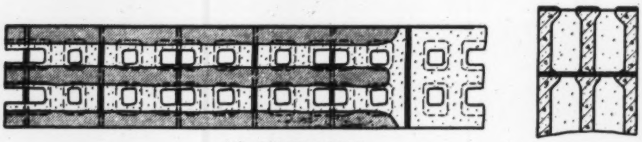
(b) GOOD

Horizontal Joints: Bedded at Face Shells and Central Web
Vertical Joints: Two Strips of Mortar with One Air Space



(c) VERY GOOD

Horizontal Joints: Bedded at Face Shells and Central Web
Vertical Joints: Three Strips of Mortar with Two Air Spaces



(d) EXCELLENT

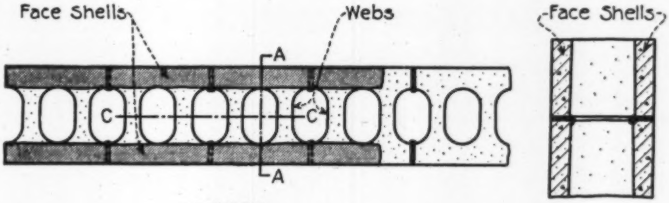
Horizontal Joints: Bedded at Face Shells and Central Web
Vertical Joints: Full Mortar Joint, no Air Spaces

FIG. 12.—Details of Horizontal and Vertical Mortar Joints in Walls made with 6-Core Tile Units laid with Workmanship Designated as "Fair," "Good," "Very Good," and "Excellent."

The shaded area in the diagrams to the left show where mortar was applied at upper bearing surfaces and at ends of units to bond them together at horizontal and vertical joints.

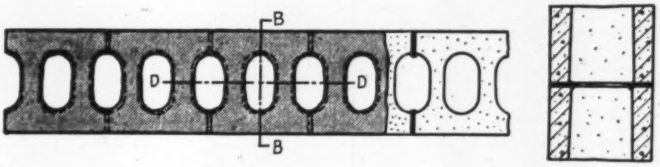
The diagrams to the right are cross-sections of walls for each type of workmenship illustrating how face shells and intermediate webs of units in adjacent courses were bonded together by the horizontal mortar joints.

altered by the variations in the cementitious material in the mortar used. For example, in the block walls the weight of the dry set mortar averaged about 2.5 lb. per sq. ft. or about 4.5 per cent of the average weight of 57 lb. per sq. ft. of wall assembly. In the tile walls, even though about twice as much mortar was used as in the block



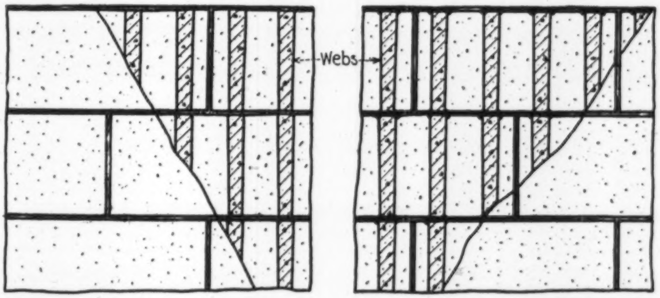
(a) GOOD
Horizontal Joints bedded at Face Shells only

Section AA



(b) EXCELLENT
Horizontal Joints bedded at Webs as well as at Face Shells

Section BB



Section CC
Webs not bedded in mortar

Section DD
Webs bedded in mortar

FIG. 13.—Details of Horizontal and Vertical Mortar Joints in Walls Made with 3-Oval-Core Block Laid with Workmanship Designated as "Good" and "Excellent."

The shaded area in Figs. (*a*) and (*b*) shows where mortar was applied at upper bearing surfaces and at ends of units to bond them together at the horizontal and vertical joints.

Sections *A-A* and *B-B* illustrate how face shells of units in adjacent courses were bonded together by the horizontal mortar joints.

Figure (*b*) and Section *D-D* of Fig. (*b*) illustrate how with "excellent" workmanship the webs of units in adjacent courses were bonded together by bedding them in mortar thereby substantially increasing the effective bearing area over that of units bedded at face shells only as shown by Fig. (*a*) and Section *C-C* of Fig. (*a*).

walls, the mortar averaged about 5.25 lb. per sq. ft. or about 9 per cent of the average weight of 58 lb. per sq. ft. of wall assembly.

*Influence of Type of Mortar on Wall Strength:*

The lower set of curves in Fig. 11 show the relations between wall and unit strength for the different types of mortar. It was found convenient to plot the curves in this manner to compensate for small variations in the strength of the units, which were not all made at the same time of the same lot of materials. It may be seen that the block walls both unexposed and those exposed to 3 hours of fire showed a gradual increase in strength as the cement content of the mortar was increased, maximum strengths being obtained with a cement content about 70 per cent of the total volume of cementitious material used (roughly 2 volumes cement to 1 volume lime). A closely similar relation is shown by the strength curve for the tile walls after $3\frac{1}{2}$ hours of fire exposure. The slight falling off in strength beyond the optimum cement content of 70 per cent in both types of wall is believed to be partly due to the lack of fatness in the mortar which influenced the character of bedding, and to a lowered efficiency of hydration of cement in the rich mixture under the dry seasoning conditions existing in the heated laboratory.

It appears from these curves that approximately the same wall strengths were obtained with mortar containing about 50 per cent cement and 50 per cent lime, which corresponds to a 1:1:6 cement-lime mortar, as with the portland-cement mortar used as standard in this investigation which contained 87 per cent cement and 13 per cent lime. This may seem rather surprising when it is considered that the portland-cement mortar is over twice as strong as the cement-lime mortar when the two are tested separately. However, it appears that in these walls of concrete masonry units other factors besides potential mortar strength come into play. Among these may be mentioned the inherent uniformity of dimensions and the comparatively large size of the concrete masonry units themselves. These factors in turn influence the number, thickness and uniformity of the mortar joints, thereby greatly reducing the influence of mortar strength on the strength of the wall assembly.

It is interesting to note from the strength curves for the block walls, for which strengths of both before and after fire exposure were available, that the ratio of wall strength after fire exposure to strength before fire exposure was practically constant at about 60 per cent regardless of variations in the cement-lime proportions of the 1:3 mortar. It would appear from this result and from the fact that closely similar strengths were obtained with mortars ranging from 50 to 87 per cent in cement content (1:1:6 to 1:0.15:3.0) that the wall strengths in the various groups of tests herein reported would have been but little affected if laid up with any mortars falling within this range of cement content.

### EFFECT OF WORKMANSHIP—GROUP 8

The 8 walls in this group, 4 of 3-oval core block (Fig. 8) and 4 of 6-core tile units (Fig. 9) were laid up with the same type of mortar to study the influence of variations in the character of both the horizontal and vertical mortar joints on fire endurance and strength.

Figure 12 gives details of the 4 walls laid up with units of 6-core tile design and with workmanship designated as "fair," "good," "very good" and "excellent" depending upon the type and combination of horizontal and vertical joints used. *Horizontal* mortar joints were of two types. In one the mortar was applied to the face shells only, and in the other to the central longitudinal web as well as the two face shells. *Vertical* mortar joints were of three types: one type consisted of two strips of mortar with one air space, another of three strips of mortar with two air spaces, while the third type consisted of full mortar joints.

TABLE V.—FIRE ENDURANCE AND STRENGTH OF WALLS LAID UP WITH DIFFERENT TYPES OF MORTAR JOINTS.

Both block and tile units were made from calcareous sand and gravel graded from 0 to $\frac{3}{8}$ in. with a constant fineness modulus of 4.0. Fine aggregate was graded from 0 to the No. 4 sieve, coarse aggregate from the No. 4 sieve to $\frac{3}{8}$ in. The mix was approximately 1:8 by volume of cement to dry rodded volume of mixed aggregate. See Figs. 12 and 13 for details of joints.

| Character of Workmanship | Quantity of Dry Set Mortar, lb. per sq. ft. of Wall Area | Fire Endurance Period, minutes | Ratio of Strength of Wall Before and After Fire to Original Strength of Units, per cent | |
|---|---|---|---|---|
| | | | Before Fire | After Fire |
| WALLS OF 6-CORE TILE UNITS | | | | |
| Fair | 3.95 | 143 | .. | 20 |
| Good | 5.25 | 155 | .. | 32 |
| Very good | 5.75 | 156 | .. | 33 |
| Excellent | 6.10 | 161 | .. | 33 |
| WALLS OF 3-OVAL-CORE BLOCK UNITS | | | | |
| Good | 2.50 | 153 | 50 | 29 |
| Excellent | 3.60 | 160 | 70 | 51 |

Figure 13 gives details of the two pairs of walls that were laid up with 3-oval core block with workmanship designated as "good" and "excellent" depending upon the type of horizontal mortar joint. The vertical joints were the same for both types of workmanship. These walls were made in pairs so that strength tests could be made both before and after fire exposure.

All walls were laid up with 1:3 portland cement mortar plus 15 per cent of hydrated lime by volume of cement in a workmanlike manner with just enough mortar to provide the horizontal and vertical joints which averaged $\frac{5}{16}$ in. in thickness.

The results of the tests are given in Table V.

*Influence of Type of Mortar Joints on Fire Endurance:*

From Table V it may be seen that the fire endurance period of walls made with tile units increased a total of 18 minutes as the type of the mortar joints was varied to correspond with the different character of workmanship ranging from "fair" to "excellent." The comparatively large difference of 12 minutes between the walls laid up with "fair" and "good" workmanship appears to be directly due to the protection afforded by the additional mortar used in the horizontal joints at the intermediate central web. The small difference in the fire endurance period of 6 minutes between the walls of "good," "very good" and "excellent" workmanship indicates that the various types of vertical joints, both the hollow and full mortar joints, had but little influence on the fire endurance period. This is not surprising when the small additional quantity of mortar used is considered. In general the fire endurance period increased progressively as the weight of the dry set mortar per square foot of wall increased, which is in harmony with the results obtained in group 4.

The fire endurance period of the walls of block units increased a total of 7 minutes as the character of workmanship was improved from "good" to "excellent." This improvement in performance was due directly to the additional mortar used to bed the transverse webs.

*Influence of Type of Mortar Joints on Wall Strength:*

The ratios of wall strength to strength of unit are given in the last two columns of Table V.

In the case of the tile walls the ratio of the strength of wall after fire exposure to the original strength of the units increased from 20 per cent for the wall of "fair" workmanship to 32 per cent for that of "good" workmanship. The substantial increase in this ratio is the direct result of bedding the intermediate central web of the tile units. Although the additional mortar used amounted to 1.30 lb. per sq. ft. of wall and represented an increase of about 33 per cent over that required for "fair" workmanship, it was more than justified as its use resulted in an increase of about 60 per cent in wall strength. This increase was somewhat greater than the corresponding increase in the area bedded and indicates that the intermediate web of the units in the wall was less affected by fire exposure than the exposed face shell. The practically constant ratios of wall strength to original strength of unit for the tile walls of "good," "very good," and "excellent" workmanship indicate that their compressive strengths after fire exposure were but slightly influenced by variations in the nature of the vertical joints, which ranged from hollow to full mortar joints, and that the additional mortar used did not augment the strength of these walls.

In the case of block walls the walls were made in duplicate, one of each pair being tested for strength without fire exposure and one after fire exposure. From the ratios of wall to unit strength in Table V it may be seen that the effect of bedding the webs of the blocks, the method designated as "excellent" workmanship in Fig. 13 (*b*), was to increase the strength of walls tested without fire exposure about 40 per cent and the strength of walls tested after fire exposure about 75 per cent. The 40-per-cent increase in the strength of the unexposed walls was roughly proportional to the increase in the area bedded. The much greater increase (75 per cent) in the strength of the exposed walls shows the marked advantage gained with the "excellent" workmanship as compared with the "good" and which resulted from the more effective use of the comparatively sound interior and unexposed portions of the wall. These results show that had the "excellent" workmanship been used in laying walls of 3-oval core block in the other groups of tests, much higher wall strengths would have been obtained both before and after fire exposure than were obtained with the "good" workmanship adopted as standard for the investigation.

### DURATION OF MOIST CURING OF UNITS—GROUP 9

The influence of curing was determined from the performance of 3 walls made from 3-oval core block identical as to cement content, type and grading of aggregate but differing as to the duration of the moist curing period. The units were made from calcareous sand and gravel, using the same grading and mix as used in group 4.

Immediately after molding, the units were removed to a room where they were moist cured for 1, 5 and 14 days at a temperature of 70° F. and in a saturated foggy atmosphere obtained by water sprays from atomizers. During the moist curing period, a slight increase in weight of blocks due to absorption of moisture was observed. A rapid loss in weight occurred when the units were removed from the moist room and loosely piled in the laboratory for drying in freely circulating air at a temperature and humidity ordinarily maintained in a heated building. After storage for 9 days under these conditions following their removal from the moist room, they were laid into walls all of which were seasoned for the same period and under identical conditions before exposure to fire.

*Influence of Curing of Units on Fire Endurance:*

The fire endurance periods were 149, 152 and 153 minutes, respectively, for 1, 5 and 14 days curing in moist room. The difference of 4 minutes for the entire group is so small as to warrant the conclusion that, from the standpoint of fire endurance the increased hydration of cement with longer curing as carried out in these tests is a factor of minor importance with ordinary 1:8 mixes. Except for the tests in this group, all units used in this investigation were moist-cured for 5 days, which period was adopted as a standard at the beginning of the work. In view of data available from other tests with tamped concrete it does not appear that the general indications from these fire endurance tests for different moist curing periods would have been greatly altered if mixes either richer or leaner than the constant 1:8 mix used in this group had been adopted.

*Influence of Curing on Strength of Units and Walls:*

The strength of the units increased with duration of moist curing, the values being 1320, 1430, and 1880 lb. per sq. in. of gross sectional area respectively for the 1, 5 and 14-day curing periods. This substantial increase in strength of the units was reflected in a corresponding increase in strength of the walls after fire exposure, which showed ultimate loads of 400, 430, and 630 lb. per sq. in. of gross sectional area respectively. It is interesting to note that the ratio of the strength of the wall after fire exposure to the original strength of the units was practically constant, varying from about 30 to 33 per cent.

### COAL AND COKE IN CINDER AGGREGATE—GROUP 10

The 9 walls in this group were made with 3 different designs of units each typical of units having relatively thick, medium, and thin face shells and webs. The three designs were 2-core block, with $2\frac{1}{8}$-in. face shells and $1\frac{5}{8}$-in. webs (Fig. 8), 3-oval core block with $1\frac{1}{2}$-in. face shells and $1\frac{1}{8}$-in. webs (Fig. 8) and 6-core tile with both face shells and webs nominally 1 in. thick (Fig. 9).

The cinder aggregate used as the base material from which the units for this group of tests were made consisted of a very good quality of well-burned soft coal cinders from a large power plant, crushed and screened to three sizes designated as fine (0 to No. 4 sieve), medium (No. 4 to $\frac{3}{8}$ in.) and coarse ($\frac{3}{8}$ to $1\frac{1}{2}$ in.). In general the combustible content of the various gradings of the crushed cinders averaged 12 per cent, of which 10 per cent was in the form of coke and 2 per cent in the form of soft coal.

The soft coal, hard coal, and coke breeze, which were added in varying amounts and proportions as described below to increase the combustible content of the base cinders, ranged from dust to lumps that would just pass a ½-in. sieve. The larger lumps of raw combustible materials were, therefore in a much more concentrated form and of somewhat greater size than the largest combustible particles that would normally be found in screened cinders.

*Tests of Walls of 2-Core Cinder Blocks:*

Two walls of 2-core block made from cinders containing approximately 16 and 33 per cent of combustible materials were tested. The combustible material contents were obtained by adding the desired amounts of a mixture of 1 part soft coal, 1 part hard coal, and 2 parts of coke breeze to the base cinders which alone contained 12 per cent of combustible matter. The cinders were graded from 0 to ½ in. by combining 4 parts of fine, 2 parts of medium, and 1 part of coarse cinders. This cinder mixture had the same fineness modulus (4.5) as the mixture of combustibles added.

The blocks were made from a 1 : 7 mix by volume of cement to dry rodded volume of the mixture of cinders and combustibles. As the result of good grading, mixing, tamping, and curing, the block were of excellent quality as to appearance, strength and denseness of concrete. With the mixture used, each particle, combustible or otherwise, was completely coated with a layer of cement paste and protected from direct contact with oxygen during fire exposure. In addition, the general denseness of the concrete served to retard the entrance of oxygen.

These tests showed not only that the fire endurance periods of the two walls were practically constant (199 and 200 minutes) but also that their load-carrying ability after 3½ hours of fire exposure was substantially the same in spite of the fact that the total combustible content of the cinders in one wall was twice that in the other. The wall of cinders containing 33 per cent of combustible material showed a slightly higher ratio of strength of wall to original strength of unit (39 per cent) than the wall with 16 per cent of combustible material (34 per cent). It appears therefore that the fire endurance period and load-carrying ability of walls constructed of well-made cinder concrete blocks of the design and cement content used in these tests is not materially influenced by combustible matter consisting of a mixture of about equal amounts of unburned coal and coke up to one-third of the dry weight of the cinders used as aggregate.

*Tests of Walls of 6-Core Cinder Tile:*

Five walls of 6-core tile units were tested to determine the relative influence of combustible matter in cinders in the form of coke as distinguished from combustible matter in the form of soft coal. Accordingly one wall, used as a basis of reference, consisted of tile made from the base cinders without the addition of either coal or coke. In this case, however, the base cinders contained approximately 14 per cent of coke and 2 per cent of soft coal, a total combustible content of about 16 per cent. The other four walls consisted of tile made from base cinders to which either soft coal or coke breeze were added to increase the total combustible content of the mixture to about 24 and 32 per cent. Except for combustible content, the cinder tile in the various walls were closely comparable having but minor variations in grading and cement content. The mixture of cinder, coke, and coal aggregate was graded from 0 to $\frac{3}{8}$ in. to a fineness modulus averaging about 3.75. The mix was 1:7.0 by volume of cement to dry rodded volume of the mixture of cinders and combustible matter.

TABLE VI.—EFFECT OF COKE AND SOFT COAL ON FIRE ENDURANCE AND STRENGTH OF 8-IN. WALLS MADE OF 6-CORE TILE UNITS.

| Aggregate | Total Combustible Content, per cent by Dry Weight of Cinders | Cement Content | | Average Weight of Air-Dry Units, lb. | Fire Endurance Period, minutes | Ultimate Strength, lb. per sq. in. Gross Area | | Ratio of Wall Strength to Original Strength of Units, per cent |
|---|---|---|---|---|---|---|---|---|
| | | Units per Sack | lb. per Unit | | | Units Unexposed to Fire | Wall after $3\frac{1}{2}$ hr. Fire Exposure | |
| Base Cinders | 16 | 38.5 | 2.44 | 14.9 | 150 | 700 | 185 | 26 |
| Base Cinders plus 9 per cent coke | 24 | 35.2 | 2.67 | 14.4 | 148 | 890 | 325 | 36 |
| Base Cinders plus 18 per cent coke | 32 | 36.7 | 2.46 | 13.3 | 134 | 750 | 170 | 22 |
| Base Cinders plus 9 per cent coal | 24 | 36.4 | 2.58 | 13.5 | 155 | 740 | 136 | 18 |
| Base Cinders plue 18 per cent coal | 32 | 34.8 | 2.70 | 13.7 | 165 | 740 | 188 | 25 |

Table VI shows that the fire endurance period *decreased* 10 per cent (from 150 minutes to 134 minutes) with the addition of 18 per cent of coke but *increased* 10 per cent (from 150 minutes to 165 minutes) with the addition of 18 per cent of soft coal. It is believed that the opposite effects of the coke and coal may be explained as follows: Coke consists almost entirely of fixed carbon (12 to 14 per cent ash), is a fairly good conductor of heat, and is fairly stable when heated to fire temperatures. By comparison, cinder material is a poor conductor of heat. The addition of coke to the cinders can, therefore, be expected to gradually increase the transmission of heat and decrease the fire endurance period as in these tests. On the other hand, soft coal, which contains a large proportion of hydrocarbons, is greatly altered and subject to destructive distillation when heated to fire temperatures. The physical and chemical changes are accompanied by an absorption of heat proportional to the coal present. The addition of soft coal to cinders can therefore be expected to gradually decrease the transmission of heat and increase the fire endurance period as occurred in these tests.

Comparison of the ratios of walls strength to strength of unit given in Table VI fails to show that a definite relation exists either between the coke, coal, or total combustible content of the cinders and the strength of the walls after 3½ hours of fire exposure. Although the ratios varied widely from 18 to 36 per cent, the average value of 25 per cent for all tests agrees very well with the ratio of 26 per cent obtained with walls and units of base cinders alone. It is significant that even on the basis of the lowest ratio obtained (18 per cent) these walls of 6-core tile units having a strength of 700 lb. per sq. in. and made from cinders containing from 16 to 32 per cent combustible material in various forms were able to carry the normal working load of 80 lb. per sq. in. of gross sectional area during and after 3½ hours of fire exposure with a margin of safety of at least 50 per cent. On the basis of the average ratio of 25 per cent, the margin of safety was about 120 per cent.

*Tests of Walls of 3-Oval-Core Block:*

Two walls of 3-oval-core block were constructed, one consisting of units made entirely of cinders (containing 12 per cent of combustible material, largely as coke) the other of units made entirely of coke breeze. The units of both types were closely comparable with but minor variations in the grading (0 to ⅜ in.) and cement content. For the cinder units the fineness modulus was 3.25, the mix 1:7.0 and the cement content 4.6 lb., while for the coke breeze the values were 3.50, 1:7.5 and 4.4 lb., respectively.

The cinder wall had a fire endurance period of 163 minutes as compared with 110 minutes for the coke breeze wall. Thus the adverse effect of increasing the quantity of coke in cinders on the fire endurance period of the 6-core tile walls was confirmed in these tests with 3-oval core block walls but which had a much greater range in coke content than the tile.

Since practically identical ratios of wall strength to original strength of unit (32 and 33 per cent for the cinder and coke breeze respectively) were obtained, it appears that the cinder aggregate can be entirely replaced with combustible material in the form of coke without materially affecting the load-carrying ability of the wall either during or after 3 hours fire exposure.

In view of the substantial load-carrying ability of all of the 9 block or tile walls in this group irrespective of variations in the nature and amount of the combustible material in the aggregate, it may be of interest to compare results on the basis of the percentage of combustible material in the concrete. The all coke breeze blocks which weighed 23.5 lb. in the air-dry condition contained about 5.3 lb. of hydrated portland cement, 18.2 lb. of coke of which about 2.2 lb. was ash and 16 lb. carbon. This corresponds to 68 per cent of combustible and 32 per cent of incombustible matter in the block or about 2 parts of combustible to 1 part of incombustible matter. In the case of the cinder units containing coal and coke, the maximum combustible content computed as above varied from 25 to 27 per cent by weight or about 1 part combustible to 3 parts of incombustible matter. The explanation of the substantial load-carrying ability of the all coke breeze wall with its high proportion of combustible matter must lie in the nature of the combustible matter (fixed carbon) as pointed out above and the protection afforded by the cement paste to the coke particles. In the case of the walls of cinder units of lower proportion of combustible matter in the form of coal and coke, the stability appears to be largely due to the protection afforded by the cement paste and the much larger proportion of incombustible matter present.

In general, the principal effect of the fire was to soften and convert the exposed face of each wall into a layer of ash from ¼ to ⅜ in. thick which served to protect the combustible matter and concrete underneath from the direct intensity of the fire. Unburned coke and coal particles could frequently be found as close as ¼ in. from the exposed face after the fire exposure. Although these had been raised rather early in the test to temperatures well above the kindling point, they lacked the oxygen necessary to support active combustion of the carbon either during fire exposure or after the incandescent face of the panel was exposed directly to the air when the wall was withdrawn from the furnace.

### MIXTURE OF TWO WIDELY DIFFERENT AGGREGATES—GROUP 11

The tests in this group were made on three similar walls constructed of 3-oval core block of variable composition obtained by the use of different proportions of two aggregates having widely different physical and mineral characteristics. The two aggregates used were the calcareous sand and gravel and the Haydite used in Group 1, both graded from 0 to $\frac{3}{8}$ in. One wall consisted of units made entirely from sand and gravel, graded to a fineness modulus of 4.50, another entirely from fine and coarse Haydite graded to a fineness modulus 4.00, while the third panel consisted of units made from a mixture of equal volumes of these two aggregates (fineness modulus 4.25). While the mix for the different aggregates varied from approximately 1:7 to 1:8 the cement content in the different units was practically constant throughout and averaged 4.50 lb. per unit.

The fire endurance periods and strength of units and walls for these tests were as follows:

| Aggregate | Fire Endurance Period, minutes | Original Strength of Units, lb. per sq. in. Gross Area | Strength of Wall After 3-hour Fire Exposure, lb. per sq. in. Gross Area | Ratio of Wall Strength to Original Strength of Units, per cent | Total Expansion of Walls (6 ft. high) during Fire Exposure, in. |
|---|---|---|---|---|---|
| All gravel............. | 141 | 1860 | 460 | 24 | 0.38 |
| Mixture............... | 157 | 1390 | 495 | 36 | 0.24 |
| All Haydite........... | 177 | 975 | 420 | 48 | 0.11 |

It is seen from this table that the fire endurance period, the strength of the walls and the strength of the individual units, and the total expansion during fire exposure of the walls made from the mixture of two widely different aggregates are approximately proportional to the volume of each type of aggregate used. In reference to fire endurance, these results are in harmony with those obtained in the second set of tests in group 10 where cinder aggregates and various mixtures of coke and soft coal were used.

### TESTS WITH FURNACE TEMPERATURES BELOW AND ABOVE STANDARD—GROUP 12

The three identical walls in this group were made with 6-core tile from 0 to $\frac{3}{8}$-in. calcareous sand and gravel aggregate (fineness modulus 4.1) with a 1:8 mix. One of the walls was exposed to fire with furnace temperatures maintained in accordance with the standard furnace control curve followed in all the other groups of tests (Fig. 4). The other two walls were exposed to fire with furnace temperatures 150° F. above and below the standard curve throughout the test. The three walls were kept exposed to the fire for 3 hours and tested for ultimate load after cooling to room temperature. The results of these tests are as follows:

| Temperatures | Fire Endurance Period, minutes | Strength of Wall After 3-hour Exposure, lb. per sq. in. Gross Area | Ratio of Wall Strength to Original Strength of Units, per cent |
|---|---|---|---|
| 150° F. below standard.................................... | 166 | 470 | 37 |
| Standard.............................................. | 155 | 452 | 35 |
| 150° F. above standard.................................... | 145 | 435 | 34 |

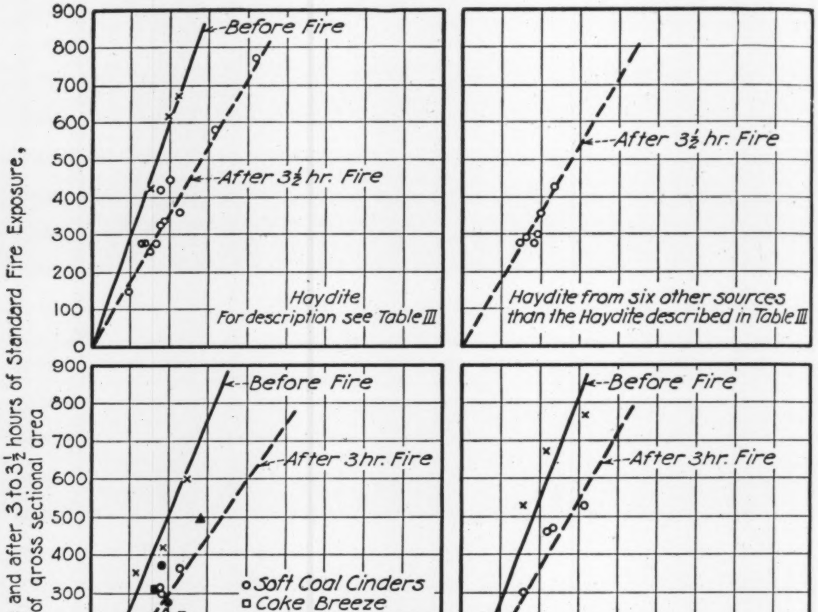


FIG. 14.—Relations Between Original Strength of Blocks made with Different Types of Aggregate and Strength of 8-in. Walls After 3 to 3½ Hours Fire Exposure.

**In general, blocks made with various gradings of aggregate and with a wide range in cement content.**

**Walls laid up with 1:3 portland-cement mortar plus 15 per cent hydrated lime by volume of cement.**

**Blocks bedded in mortar applied at face shells only as shown in Fig. 13 (*a*).**

**Walls exposed to fire allowed to cool to room temperature for 24 hours before loading to failure.**

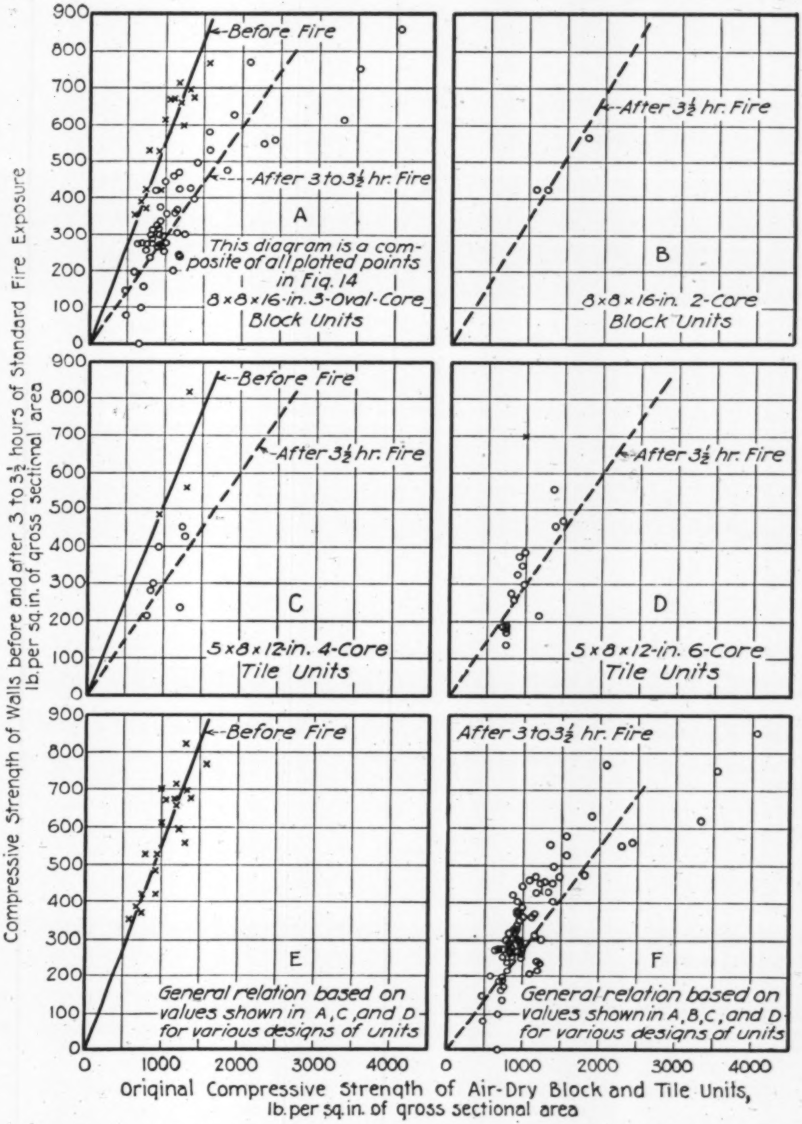


FIG. 15.—General Relations Between Original Strength of Units of Various Designs and Strength of 8-in. Walls Before and After 3 to 3½ hours Fire Exposure.

Plotted values include all tests made with different types and gradings of aggregate and with different cement contents.

Walls laid up with 1:3 portland-cement mortar plus 15 per cent hydrated lime by volume of cement.

Blocks bedded in mortar applied at face shells only as shown in Fig. 13 (*a*).

Tile units bedded in mortar applied at face shells and central web as shown in Fig. 12 (*b*).

Walls exposed to fire allowed to cool to room temperature for 24 hours before loading to failure.

It is seen from the above table that the fire endurance period was decreased or increased about in proportion to the variation of the furnace temperatures above or below the standard. Differences in strength of walls after the 3-hour fire exposure are seen to be quite small and indicate that the intensity of furnace temperatures within these limits had little effect on the strength. While duplication of standard furnace temperatures was one of the important features of this entire investigation, the results of this group of tests gives assurance that slight deviations from the standard furnace temperatures which sometimes occurred were practically insignificant in effect either from the standpoint of fire endurance period or strength of wall.

## EFFECT OF VARIOUS FACTORS STUDIED ON STRENGTH OF WALLS BEFORE AND AFTER FIRE EXPOSURE

One of the outstanding results of this investigation was the load-carrying ability and safety of the walls, before, during and after severe fire exposure which was repeatedly demonstrated with the many variables of composition and design of the units and of the type of mortar and workmanship employed.

The data on the strengths of 8-in. walls, comparable as to type of mortar and workmanship, is presented in condensed form in the various graphs in Figs. 14 and 15 which are arranged to show the influence of strength of unit, type of aggregate, and design of unit on the load-carrying ability of the walls both before and after 3 and $3\frac{1}{2}$ hours of standard fire exposure. These graphs are based on the strengths of 98 walls, 20 of which were tested to determine the strength before fire exposure for comparison with the strength of the remaining 78 panels tested after fire exposure.

### *Relation Between Strength of Wall and Original Strength of Unit:*

In Fig. 14, the data from these tests are arranged to show separately the tests on the different types of aggregates. All the units were 8 by 8 by 16 in. of 3-oval core block laid up with 1:3 portland-cement mortar (plus 15 per cent hydrated lime) with type of workmanship designated in Fig. 13(*a*) as "good". It will be seen from a study of the diagrams in this figure that the compressive strength of the walls tested without exposure to fire or after exposure to fire bears a straight-line relationship to the original compressive strength of the block. The establishment of this relationship was one of the most useful and important developments of the strength tests. Its significance will be appreciated when the wide limits of strength of unit, type and grading of aggregate, design of unit and cement content represented are considered.

In diagram *A* of Fig. 15 the results of all six diagrams of Fig. 14 are superimposed. While there is some scattering of the points representing the tests after fire exposure it is seen that in each case the results of the entire group of tests are quire well represented by a single line.

Diagrams *B*, *C*, and *D* of Fig. 15 show the data for tests on units of other types. In these tests all the data for a given type unit, regardless of aggregate and cement content or grading, are included in the single diagram. The mortar and workmanship in these tests were the same as in the tests shown in Fig. 14. These diagrams also show the same regularity of relationship between strength of wall and strength of unit.

In diagrams *E* and *F* of Fig. 15 all the data from diagrams *A*, *B*, *C*, and *D* are replotted to separate the tests made on the walls which have not been exposed to fire from those subjected to the 3 or 3½-hour fire test. As noted above in reference to diagram *A*, the agreement of the plotted points with the straight line is somewhat better for the tests of walls which have not been exposed to fire, but in either case a marked tendency to rectilinear variation of wall strength with strength of unit is very definite.

*Comparison of Types of Aggregate:*

The data in Fig. 14 are useful in comparing the several types of aggregates as to their effect upon the load-carrying ability of 8-in. walls after 3 or 3½-hour fire exposure. If it is desired to compare the strengths of walls built from units of a given initial strength after this fire exposure, values can be taken directly from the proper curves of Fig. 14. For example, for units having an initial strength of 700 lb. per sq. in. gross area (the strength required by various standards) the strength of walls after 3 to 3½-hour fire exposure for various aggregates would be as follows: Haydite and limestone, 250 lb.; cinders, blast-furnace slag, crushed common brick, crushed fire brick, and coke breeze, 210 lb.; calcareous sand and gravel, 190 lb. per sq. in. While these values are of interest for comparative purposes, they are of great practical importance in showing that in general 700-lb. units from the above aggregates produced walls having strengths after severe fire exposure of from two to three times the specified working load of 80 lb. per sq. in. of gross area.

It will be noted in the lower right-hand diagram of Fig. 14 that the walls containing the highly siliceous aggregates were appreciably lower in strength after fire exposure for a given original strength of unit than the walls made with the other aggregates. Moreover, in the case of the two walls made with 700-lb. units the results after fire exposure were erractic. The strength of the wall built with all fine siliceous aggregate (fineness modulus 2.46) and of 1:8 mix was 100 lb. per sq. in. of gross area and conformed closely to the straight-line strength graph, while the wall made with aggregate containing a large proportion of coarse particles (fineness modulus 4.25) and of a lean 1:14 mix, ceased to carry the 80-lb. working load after 93 minutes of fire exposure. Since the strengths of the other four walls in this group conformed closely to the straight line through the plotted points, even though the same coarse grading (fineness modulus 4.25) was used in two of the walls but in richer mixes (1:7.5 and 1:3), it appears that the erratic performance of the one wall was due to a critical combination of coarse grading and lean mix, which resulted in a comparatively large loss of bond between the highly siliceous aggregate particles as they expanded with exposure to fire. These results with the highly siliceous aggregates indicate, first, that coarse gradings and excessively lean mixes should be avoided and, second, that in general the original strengths of the units must be well above 700 lb. if walls equal in load-carrying ability after fire exposure to those made from the other aggregates are to be obtained.

The data presented in Fig. 14 can be utilized to select the strength of unit necessary to produce any given wall strength after fire exposure. In order to obtain a wall with the highly siliceous gravels, comparable in strength, for example, with a wall of Haydite units, having original strengths of 700 lb. per sq. in., the diagrams show that it will be necessary to use units of about 1200 lb. per sq. in. initial strength. In each of these cases the wall after 3 or $3\frac{1}{2}$-hour fire exposure would have a strength of 250 lb. per sq. in. gross area. It can be seen, therefore, that the problem of selecting a wall for a given fire exposure and strength requirement is not alone one of selection of aggregate, but rather must involve also the matter of economy as affected by cement content, grading, or other factors. For example, by reference to the lower curve of Fig. 7, it will be seen that to obtain a strength of 1200 lb. per sq. in. with the siliceous sand and gravels, about the same amount of cement per block is required as for Haydite block of 700 lb. per sq. in. initial strength.

### *Comparison of Design of Unit:*

A comparsion of diagrams *A*, *B*, *C*, and *D* in Fig. 15 shows that irrespective of the type of unit used, 8-in. walls made with units of a given strength had approximately equal strengths after fire exposure.

## GENERAL COMMENTS ON THE EFFECT OF FIRE EXPOSURE

*Spalling.*—In no case was there any evidence of spalling of the concrete on either the exposed, unexposed or interior surfaces of any of the test walls either during or after fire exposure in spite of the great range in the grading of aggregate, cement content, type of aggregate, strength and denseness of the concrete, or in the various designs of units with thin, medium and thick face shells. An occasional local spalling or pitting of aggregate particles at the exposed face of the walls was found in the case of two aggregates, Haydite and air-cooled blast-furnace slag. These local effects, however, were of no consequence as they did not appear to affect in any way the load-carrying ability or fire endurance of the walls.

*Expansion.*—The thermal expansion of the walls upon heating varied with the type of aggregate. With those aggregates showing relatively high expansions, the amount of expansion increased markedly as the proportion of coarse to fine particles in the mixture increased. The expansion of the walls seemed to be relatively little affected by variations in the cement content of the concrete. Following are the approximate total expansions (average of maximum expansions observed) in inches of walls (6 ft. high) made with 3-oval core block of constant cement content (1:8 mix) during 3 to $3\frac{1}{2}$ hours of fire exposure with temperatures at the exposed face reaching 1900 to 2000° F.:

| Type of Aggregate, (Graded 0 to $\frac{3}{8}$ in.) | Total Expansion for Gradings Ranging from Fine to Coarse as Shown in Table I, in. |
|---|---|
| Cinders, blast-furnace slag, coke breeze, common brick, fire brick, and Haydite | $\frac{1}{16}$ to $\frac{1}{8}$ |
| Limestone, calcareous sand and gravel | $\frac{3}{16}$ to $\frac{3}{8}$ |
| Highly siliceous sand and gravel | $\frac{5}{16}$ to $\frac{9}{16}$ |

The differential expansion between the exposed and unexposed faces of the walls caused them to bow or deflect towards the fire side the beginning of which was observed after about 10 minutes of fire exposure. This bowing generally continued during the entire exposure period and was greatest for the walls showing the greatest thermal expansion. Because of this bowing during the test, a greater proportion of the superimposed load was carried by the face of the wall exposed to the fire. This fact adds significance to the data showing the load-carrying ability of the walls during and after exposure to fire.

It will be recalled that the data in the tabulation under group 11 showed that the total expansion of a wall made from a mixture of two widely different aggregates, which in themselves have different expansion properties, was directly proportional to the volume of each aggregate used. It can be seen from this and from the discussion above that aggregates having high expansion properties can be successfully used either by changing the grading or by using them in suitable proportions with aggregates of low expansion properties. Further studies are now under way on the effect of mixtures of aggregates of different properties on the behavior of concrete during fire exposure.

*Loss of Bond.*—One of the general effects of continued fire exposure was the impairment of the bond between the cement paste and aggregate particles. This effect was most pronounced with the highly siliceous aggregates which showed the highest thermal expansion. Although this effect was reflected in a proportionately lower wall strength after fire exposure, it is significant that equivalent load-carrying properties can easily be obtained through an increase in the original strength of the units by increasing the cement content.

*Fusion.*—With fire exposures up to 5 hours, fusion of the exposed surface of the walls occurred only to a limited extent in the case of water-cooled blast-furnace slag. With this aggregate, fusion was manifested during the first 2 hours of fire exposure with the formation of numerous light, porous growths of varying color sometimes extending $\frac{1}{2}$ in. from the exposed face of the wall. These formations did not appear to cause the occasional local spalling of the aggregate particles noted above and did not in any way appear to influence the load-carrying ability of the walls.

*Effect on Mortar.*—In general the mortar joints on the exposed face of the walls appeared to be much more affected by fire exposure than the adjacent exposed surfaces of the units. The mortar was generally softened for a depth of from $\frac{1}{2}$ to $\frac{3}{4}$ in.

## SUMMARY AND CONCLUSIONS

The relative influence of the various factors of composition, form and assembly upon the fire resistance and stability of walls of concrete masonry units from the tests conducted thus far in this comprehensive investigation may be summarized as follows:

1. *Grading.*—Tests with four widely different types of aggregate showed that, with a given cement content, the fire endurance period increased as the proportion of fine to coarse aggregate in the concrete mixture was increased in spite of the accompanying variations in the voids, weight and strength of the units. In these tests, which covered the entire range of grading possible in the practical manufacture of

concrete masonry units by the "tamped" method, the fire endurance period with the finest aggregate grading was from 15 to 20 per cent greater than with the coarsest grading.

The effects of grading upon fire endurance period, however, were insignificant in comparison with their very marked effect on strength of units and corresponding strength of walls both before and after fire exposure. The proportion of fine to coarse aggregate should in general be based upon considerations of strength of wall rather than upon fire endurance.

2. *Cement Content.*—Tests with units made with 3 different types of aggregate and from mixes ranging from lean to rich (1:14 to 1:3 by volume) showed that the fire endurance period increased directly with increase in cement content regardless of the accompanying variations in the other physical properties of the units and the type of aggregate used. The increase in fire endurance period with increase in cement content ranged from about 20 per cent to 35 per cent depending on the type of aggregate.

The strength of units and of the corresponding walls made with each type of aggregate were for all practical purposes directly proportional to the cement content. Generally speaking, doubling the cement content resulted in an increase in fire endurance period of the walls of at least 10 per cent and an increase in load-carrying ability both before and after fire exposure of about 100 per cent.

3. *Type of Aggregate.*—With a given grading and cement content, the fire endurance period of 8-in. walls was substantially influenced by the type of aggregate used. The range in the fire endurance period for 10 widely different aggregates was from 27 per cent below to 27 per cent above the average period of 152 minutes for the group. When compared on the basis of fire endurance in minutes per pound of material per square foot of wall the four aggregates showing the highest fire endurance periods were nearly twice as effective as the two aggregates showing the lowest fire endurance periods. However, the results of tests of the other factors studied, showed that by the proper selection of type of unit, cement content, grading of aggregate, wall thickness, etc., any particular requirements can be met with any of the materials tested.

4. *Design of Unit.*—In tests of units of different designs made with calcareous sand and gravel aggregates of constant grading and constant cement content, a remarkably consistent relation between the fire endurance period and the weight per square foot of wall was found. There was a marked increase in the fire endurance period with increase in weight. In general, an increase of 50 per cent in

weight doubled the fire endurance period. The relationship appears to be fundamental and to hold regardless of the manner in which the concrete was distributed in the different designs of units. It shows that the effectiveness of a given type of concrete can only be improved by increasing the weight of the unit. Preliminary information from tests along the same lines with other types of aggregate have shown a similar general relationship between weight of wall and fire endurance period. When completed, these tests will provide a rational basis for design of walls to give any desired performance.

5. *Effect of Plaster.*—The fire endurance period of 8-in. walls was increased approximately 30 per cent when finished with ½ in. of gypsum plaster on either the exposed or unexposed face and about 60 per cent when finished with ½ in. of gypsum plaster on both faces. These tests covered walls made with 2 widely different aggregates. In general the application of plaster to one or both faces of the wall resulted in an appreciable improvement in wall strength after the same or somewhat longer periods of fire exposure.

6. *Type of Mortar.*—The fire endurance period of walls of both block and tile units laid up with 5 different types of 1:3 mortar (by volume of dry rodded sand) ranging from straight lime mortar to portland-cement mortar was practically unaffected by wide variations in the cement-lime proportions.

On the other hand, the strength of the walls both before and after fire exposure increased as the cement content of the mortar increased, maximum strengths being obtained with a cement content of about 70 per cent of the total volume of cementitious material (roughly 2 volumes cement to 1 volume hydrated lime). A slight falling off in strength occurred with increase in cement beyond the optimum cement content of about 70 per cent. In general, closely similar wall strengths were obtained both before and after fire exposure with 1:3 mortars ranging from 50 to 87 per cent in cement content (1:1:6 to 1:0.15:3.0). From these results it appears that although the walls in this investigation were for the most part laid up with the 1:0.15:3.0 mortar (1:3 portland-cement mortar plus 15 per cent lime), the wall strengths obtained would have been little affected if any other mortar falling within the above range of cement contents had been used.

7. *Type of Joints.*—The type of joint used in laying up the walls had a much greater influence on wall strength after fire exposure than on the fire endurance period. Improvement in both fire endurance and wall strength was obtained principally by a more complete bedding of the units at the horizontal joints rather than by a more

complete filling of the vertical joints. In the case of 6-core tile units, bedding the intermediate web as well as the two face shells increased the fire endurance period from 8 to 12 per cent, depending on whether hollow or full vertical joints were used, and increased the strength of wall after fire exposure 60 per cent regardless of the type of vertical joint used. In the case of 3-oval-core block units, bedding the transverse webs as well as the face shells increased the fire endurance period only 4 per cent but increased the wall strength after fire exposure about 75 per cent.

8. *Curing of Units.*—Only a very slight and practically negligible increase in the fire endurance period resulted when the duration of moist curing of the units was extended from 1 day to 5 and 14 days. However, a substantial increase in the strength of the units resulted from the longer periods of moist curing and this was reflected in a corresponding increase in strength of the walls.

9. *Combustible Content of Cinder Aggregate.*—The effect of increasing the combustible content of soft coal cinder aggregate upon the fire endurance period of walls constructed of well-made cinder units was found to depend on the nature as well as the amount of the combustible material added. The fire endurance period was decreased about 10 per cent with the addition of 17 per cent of combustible material in the form of coke but increased about 10 per cent with the same addition of soft coal. As a result of these diametrically opposite effects it was found that the fire endurance period remained constant as long as about equal quantities of both coal and coke were added, even though the total combustible content was increased from 16 to 33 per cent of the dry weight of the cinders.

The tests showed that soft coal cinders may contain a mixture of unburned coal and coke up to approximately one-third of the dry weight of the cinders without materially influencing the load-carrying ability of walls constructed of well-made cinder concrete block or tile units either during or after 3 to $3\frac{1}{2}$ hours of severe fire exposure.

10. *Mixture of Two Different Aggregates.*—The fire endurance periods of walls made from units containing a mixture of two widely different aggregates, which by themselves have different thermal properties and produce walls and units of different strengths, were approximately proportional to the volume of each type of aggregate used. The strength of the wall, the strength of the individual units, and the total expansion of the walls were found to be influenced in a similar manner.

11. *Furnace Temperatures Above and Below Standard.*—In tests with furnace temperatures 150° F. above or below the standard time-

temperature curve, the fire endurance period decreased or increased in proportion to the variation of the furnace temperatures above or below the standard. Differences in strength of the walls after 3 hours of fire exposure were quite small and indicate that the intensity of furnace temperature within these limits had little effect on strength.

12. *Wall Stability.*—One of the outstanding results of this investigation was the substantial load-carrying ability and safety of the walls before, during, and after severe fire exposure which was repeatedly demonstrated notwithstanding the many variables of composition and design of the units and of the type of mortar and workmanship employed in their construction.

13. *Relation Between Strength of Wall and Strength of Unit.*—One of the most useful and important developments of the strength tests was the establishment of a straight-line relationship between the original compressive strength of the units and the compressive strength of the walls before and after exposure to fire. This relationship shows that the strength of the wall is approximately proportional to the strength of the units over a wide range in strength of unit, type and grading of aggregate, design of unit and cement content.

14. The various relationships and data established by these tests provide basic information for the manufacture of concrete masonry units, from a wide range of available materials, which will produce walls meeting any given requirements as to fire endurance and strength.

[For Discussion on Masonry Building Units, see page 681.—ED.]

# TESTS OF THE STABILITY OF CONCRETE MASONRY WALLS

By F. E. Richart,[1] P.M. Woodworth[2] and R. B. B. Moorman[3]

## Synopsis

This paper gives results of tests of 60 concrete masonry walls, 6 ft. long and 9 ft. 6 in. high; 42 wallettes, or small walls, 32 in. long and 4 ft. high, and accompanying tests of building units and materials. The walls were tested in compression, axial and eccentric, and in flexure. From the results of the tests, several fairly definite relations were established. The compressive strength of large walls was shown to be quite closely dependent upon the compressive strength of the units used therein, and the average ratio of strength of wall to that of unit was found to be 0.53. The ratio of compressive strengths of large and small walls was found to be fairly constant, with an average value of 0.91. This would seem to establish the propriety of using wallette tests as indicative of the strength of wall construction, using the reduction factor found. Eight-inch walls with face-shell mortar beds were found to have four-fifths of the compressive strength of similar walls with full mortar beds. The effect of other variables included in the investigation, such as type of aggregate, strength of mortar and thickness of walls was found to be less definite. Composite walls of face brick and concrete units showed satisfactory behavior and strength.

The average compressive strength of all large walls varied from 335 to 850 lb. per sq. in. The flexural strength of walls, which failed at the junction of units and horizontal mortar joints, varied from 18 to 50 lb. per sq. in. Walls under eccentric loads applied at the edge of the middle third of the thickness carried loads three-fourths as great as were carried when the load was centrally applied.

## Object of Investigation

In the development of standard specifications for masonry building units it is necessary to know what relation exists between the strength properties of building unit and mortar and the strength and stability of walls in which they are used. Except for tests of pilasters and of walls made recently in connection with fire tests, there is little information on the strength of story-height construction with concrete masonry. The following investigation was designed to study those factors which influence masonry strength and to give further indication as to the essential properties to be specified for hollow concrete building units. The matter becomes of immediate importance to the Society because of its interest in the preparation of specifications for hollow concrete units, and this progress report of the wall investigation is made now with the object of placing before the Society the principal results of the investigation, obtained from tests carried on since the beginning of the year.

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*Acknowledgments:*

The investigation described herein was carried on at the Materials Testing Laboratory, University of Illinois, in cooperation with the Portland Cement Association and the Concrete Masonry Association. The work was done as a regular project of the Engineering Experiment Station under the administrative direction of Dean M. S. Ketchum, Director of the Station, and Prof. M. L. Enger, Head of the Department of Theoretical and Applied Mechanics. The two associations were represented by an Advisory Committee consisting of W. D. M. Allan, chairman, Nolan Browne, D. R. Collins and Benjamin Wilk. Members of this committee cooperated in the outlining of the investigation and assisted in providing facilities for the work.

Various materials were donated for use in the investigation as noted in the section on materials. These are essentially donations to the Research Program of the Concrete Masonry Association and as such have been acknowledged elsewhere. Acknowledgment is due to the Nelson Concrete Culvert Co., Champaign, Ill., E. P. Shapland, secretary, who gave excellent cooperation in the manufacture of building units used.

## OUTLINE OF TESTS

The tests were planned primarily to furnish information on the behavior under load of wall panels of sufficient size to represent building construction; with this in view a panel 6 ft. long and about 9 ft. 6 in. high was chosen. Small panels, or wallettes, 32 in. long and 48 in. high (having about the same ratio of width to height as the large walls) were also used to determine whether or not they could be established as representative test specimens for masonry walls, and, if so, to determine the correction factor to be used with wallette strengths to make them directly comparable with the values from the large panels. The main group of test panels was planned for a compression test with a uniformly applied axial load. Four lots of walls were also tested to furnish information on the strength of walls eccentrically loaded. To secure data on the flexural strength of the various types of walls, a large panel of each lot was subjected to a bending test.

The general schedule of walls tested is listed below. Generally, two small walls and two or three large ones (one tested in bending) are represented by each item:

| | |
|---|---|
| 1. Type of Aggregate...<br>(Lots 1 to 8) | Units made of cinders, Haydite, sand-gravel, limestone and crushed air-cooled slag. First 3 used with 2 strengths of unit and 2 forms of mortar bed. |
| 2. Strength of Units....<br>(Lots 1 to 3, 6 to 8, 10 to 11) | Units of Haydite, cinders and sand-gravel concrete made in strengths of 550 to 1550 lb. per sq. in. |
| 3. Type of Mortar......<br>(Lots 1, 3, 8 to 11) | Two mortars, $1:1:4\frac{1}{2}$, cement, lime and sand, dry rodded, and 1:3 cement mortar with addition of 10 per cent of lime by weight of cement. |
| 4. Type of Mortar Joint.<br>(Lots 1 to 3) | Walls of cinder, Haydite and sand-gravel concrete units laid up with full mortar beds and with mortar bed on face shells only. |
| 5. Design of Unit.......<br>(Lots 8, 12, 13) | Three types of unit in 8-in. wall........ 8 by 8 by 16-in., with 3 oval cores<br>5 by 8 by 12-in. tile, 2 square cores<br>$3\frac{1}{2}$ by 8 by 12-in. tile, 2 square cores |
| 6. Wall Thickness......<br>(Lots 2, 3, 14, 15) | Two thicknesses of wall made with 8 by 8 by 16-in. and 8 by 12 by 16-in. units, 3 oval cores, with Haydite and sand-gravel units. |
| 7. Composite Walls.....<br>(Lots 16, 17) | Walls with 4-in. of face brick and 8 by 8 by 16-in. backup block of Haydite and cinder concrete. Header course every sixth brick course. |
| 8. Eccentric Loading...<br>(Lots 18 to 21) | Walls of Haydite, cinder and sand-gravel block and sand-gravel tile units. Loaded with eccentricity one-sixth of wall thickness. |

## MATERIALS AND MANUFACTURE OF UNITS

All of the concrete masonry units required by the foregoing outline of tests, with the exception of the tile (lots 12, 13 and 21) were made at the plant of the Nelson Concrete Culvert Co., Champaign, Ill., under normal conditions of plant manufacture.

*Aggregates.*—Of the five types of aggregates used, the Haydite, donated by the Western Brick Co., Danville, Ill., and the sand and gravel, from the Lincoln Sand and Gravel Co., Lincoln, Ill., were of the stock in regular use at the plant. The crushed limestone, donated by the Elmhurst Chicago Stone Co., Elmhurst, Ill., cinders from the plant of the Corn Products Co., Argo, Ill., and crushed air-cooled slag, donated by the Illinois Slag and Ballast Co., Chicago, Ill., were representative of materials regularly used in the production of concrete masonry units in this locality. The above aggregates were obtained in two nominal sizes, 0 to No. 4 and No. 4 to $\frac{3}{8}$ in., with the exception of the cinders, for which the larger size was No. 4 to $\frac{1}{2}$ in.

The sand and gravel used in the tile units made at Dallas, Tex., and Memphis, Tenn., were representative materials in regular use. Sieve analyses of all aggregates are given in Table I.

*Cement and Lime.*—The portland cement used in the manufacture of units and in mortar mixtures was a blend of three brands purchased on the local market. The hydrated lime used in the mortar mixes was a single brand, bought locally.

*Brick.*—The face brick used in the construction of the composite wall panels were donated by the Western Brick Co.

*Manufacture of Units.*—The units with three oval cores (8 by 8 by 16-in., and 8 by 12 by 16-in.) were made on contract at the local plant using an Anchor hopper feed, mechanical tamper and stripper machine. The concrete was mixed in a paddle mixer in batches producing 15 to 22 blocks. The materials for each batch were measured separately by means of a Johnson weighing batcher and combined at the mixer. The proportions of fine to coarse aggregates in general followed the plant practice and were designed to produce units with a fairly rough texture. The fineness modulus of the combined aggregates was 4.0 to 4.25 except for cinders in which the coarse grading produced a value of 4.70 in the mix chosen. Each batch using sand-gravel, limestone and slag was mixed 1 minute dry and 5 minutes after water was added. The Haydite and cinder aggregates were mixed 2 minutes wet with about half of the mixing water added, and 3 minutes more after the cement and the balance of the water were added. The proper consistency was determined by the plant operator, based upon the appearance of the units. Each block was tamped from 10 to 12 times with each of the tamper feet.

Table I.—Sieve Analysis of Aggregates.

| Aggregate | Percentage Retained on Sieve | | | | | | | Fineness Modulus |
|---|---|---|---|---|---|---|---|---|
| | $\frac{3}{8}$ in. | No. 4 | No. 8 | No. 16 | No. 30 | No. 50 | No. 100 | |
| Lincoln Sand | 0 | 3 | 14 | 29 | 48 | 89 | 98 | 2.81 |
| Lincoln Gravel | 8 | 89 | 98 | 99 | 100 | .. | .. | 5.94 |
| Illinois Slag, fine | 0 | 3 | 32 | 57 | 71 | 82 | 91 | 3.36 |
| Illinois Slag, coarse | 8 | 85 | 98 | 99 | 100 | .. | .. | 5.89 |
| Elmhurst Limestone, fine | 0 | 0 | 26 | 51 | 72 | 77 | 84 | 3.10 |
| Elmhurst Limestone, coarse | 0 | 27 | 87 | 89 | 91 | 92 | 93 | 4.79 |
| Argo Cinders, fine | .. | 33 | 55 | 67 | 74 | 80 | 86 | 3.95 |
| Argo Cinders, coarse | 54 | 86 | 89 | 91 | 91 | 91 | 91 | 5.93 |
| Western Haydite, fine | 0 | 1 | 11 | 43 | 62 | 73 | 80 | 2.70 |
| Western Haydite, coarse | 6 | 75 | 96 | 97 | 97 | 97 | 97 | 5.65 |
| Dallas Sand | 0 | 0 | 1 | 9 | 32 | 80 | 99 | 2.21 |
| Dallas Gravel | 18 | 87 | 99 | 100 | .. | .. | .. | 6.04 |
| Memphis Sand | .. | .. | 0 | 8 | 37 | 79 | 96 | 2.20 |
| Memphis Gravel | 8 | 71 | 96 | 99 | 100 | .. | .. | 5.74 |
| Lincoln Mortar Sand | .. | .. | 0 | 11 | 41 | 87 | 98 | 2.37 |

TABLE II.—DETAILS OF CONCRETE MIXES USED IN MANUFACTURE OF UNITS.

| Lot | Type of Aggregate | Mix by Weight |  |  | Water Added at Mixer, lb. | Moisture in Aggregate, per cent |  | Fineness Modulus of Combined Aggregate | Number of Units per Sack of Cement |
|---|---|---|---|---|---|---|---|---|---|
|  |  | Cement, lb. | Aggregate |  |  |  |  |  |  |
|  |  |  | Fine Damp, lb. | Coarse Damp, lb. |  | Fine | Coarse |  |  |
| No. 1... | Cinders.................. | 94 | 405 | 245 | 46.5 | 11 | 6 | 4.70 | 21 |
| No. 2... | Haydite.................. | 94 | 250 | 250 | 75.0 | 7 | 9 | 4.17 | 22 |
| No. 3... | Sand and Gravel.......... | 94 | 840 | 555 | 62.5 | 4 | 3 | 4.06 | 28 |
| No. 4... | Limestone................ | 94 | 460 | 700 | 70.0 | 11 | 4 | 4.04 | 25 |
| No. 5... | Slag..................... | 94 | 550 | 295 | 42.0 | 10 | 2 | 4.24 | 21 |
| No. 6... | Haydite.................. | 94 | 185 | 185 | 52.0 | 7 | 9 | 4.17 | 16 |
| No. 7... | Cinders.................. | 94 | 275 | 165 | 35.0 | 11 | 6 | 4.70 | 14 |
| No. 8... | Sand and Gravel.......... | 94 | 625 | 410 | 53.0 | 4 | 3 | 4.06 | 21 |
| No. 9... | Cinders.................. | 94 | 385 | 225 | 47.0 | 11 | 6 | 4.70 | 20 |
| No. 10.. | Sand and Gravel.......... | 94 | 860 | 560 | 45.0 | 4 | 3 | 4.06 | 31 |
| No. 11.. | Sand and Gravel.......... | 94 | 625 | 410 | 53.0 | 4 | 3 | 4.06 | 21 |
| No. 12[a]. | Sand and Gravel (Dallas).. | 94 | 428 | 203 | 62.0 | 4 | 2 | 3.47 | 35 |
| No. 13[a]. | Sand and Gravel (Memphis) | 94 | 215 | 400 | .... | .. | .. | 4.37 | 44 |
| No. 14.. | Sand and Gravel.......... | 94 | 900 | 600 | 77.0 | 4 | 3 | 4.06 | 19 |
| No. 15.. | Haydite.................. | 94 | 250 | 250 | 67.0 | 7 | 9 | 4.17 | 15 |
| No. 16.. | Haydite.................. | 94 | 250 | 250 | 75.0 | 7 | 9 | 4.17 | 22 |
| No. 17.. | Cinders.................. | 94 | 380 | 220 | 47.0 | 11 | 6 | 4.70 | 20 |
| No. 18.. | Haydite.................. | 94 | 195 | 195 | 58.0 | 7 | 9 | 4.17 | 17 |
| No. 19.. | Cinders.................. | 94 | 405 | 245 | 46.5 | 11 | 6 | 4.70 | 21 |
| No. 20.. | Sand and Gravel.......... | 94 | 890 | 590 | 72.0 | 4 | 3 | 4.06 | 31 |
| No. 21[a]. | Sand and Gravel.......... | 94 | 428 | 203 | 62.0 | 4 | 2 | 3.47 | 35 |

[a] Data supplied by manufacturer.

TABLE III.—PHYSICAL PROPERTIES OF CONCRETE MASONRY UNITS.

Each value represents tests of 5 units, except values of unit weight which are based on tests of 3 units.

|  | Type of Aggregate | Weight of Concrete, lb. per cu. ft. | Weight of Unit, lb. | Absorption, lb. per cu. ft. | Moisture in Unit, per cent |  | Compressive Strength, lb. per sq. in., Gross Area |  |  |
|---|---|---|---|---|---|---|---|---|---|
|  |  |  |  |  | 28 days | 60 days | 7 days | 28 days | 60 days |
| 8 BY 8 BY 16-IN. UNITS; 3 OVAL CORES; CORE AREA 37 PER CENT |  |  |  |  |  |  |  |  |  |
| No. 1.. | Cinders.......... | 92 | 33.6 | 13.8 | 6.7 | 2.8 | 440 | 730 | 865 |
| No. 2.. | Haydite.......... | 75 | 25.9 | 14.4 | 3.9 | 1.9 | 490 | 760 | 820 |
| No. 3.. | Sand and Gravel.. | 126 | 47.8 | 8.5 | 2.5 | 1.1 | 650 | 830 | 1070 |
| No. 4.. | Limestone........ | 129 | 46.4 | 13.2 | 3.0 | 2.5 | 600 | 730 | 780 |
| No. 5.. | Slag............. | 127 | 45.3 | 11.5 | 3.3 | 2.9 | 810 | 1140 | 1290 |
| No. 6.. | Haydite.......... | 77 | 27.6 | 17.8 | 8.7 | 4.5 | 630 | 950 | 1280 |
| No. 7.. | Cinders.......... | 97 | 33.5 | 14.0 | 8.0 | 3.9 | 690 | 780 | 970 |
| No. 8.. | Sand and Gravel.. | 128 | 48.8 | 7.8 | 2.2 | 1.1 | 840 | 1080 | 1230 |
| No. 9.. | Cinders.......... | 101 | 35.8 | 12.0 | 5.4 | 1.6 | 640 | 870 | 1150 |
| No. 10. | Sand and Gravel.. | 125 | 45.8 | 9.7 | 2.1 | 0.8 | 400 | 550 | 550 |
| No. 11. | Sand and Gravel.. | 128 | 49.1 | 8.2 | 3.1 | 1.8 | 980 | 1120 | 1570 |
| No. 16. | Haydite.......... | 74 | 25.7 | 15.8 | 6.1 | 2.6 | 640 | 710 | 720 |
| No. 17. | Cinders.......... | 91 | 32.6 | 14.1 | 6.3 | 2.9 | 560 | 630 | 810 |
| No. 18. | Haydite.......... | 75 | 26.9 | 13.3 | 8.0 | 3.4 | 620 | 740 | 1040 |
| No. 19. | Cinders.......... | 92 | 33.6 | 13.8 | 6.7 | 2.8 | 440 | 730 | 850 |
| No. 20. | Sand and Gravel.. | 126 | 47.2 | 7.1 | 2.8 | 1.6 | 660 | 900 | 1010 |
| 8 BY 12 BY 16-IN. UNITS; 3 OVAL CORES; CORE AREA 39 PER CENT |  |  |  |  |  |  |  |  |  |
| No. 14. | Sand and Gravel.. | 127 | 64.4 | 9.6 | 3.0 | 1.2 | 420 | 620 | 600 |
| No. 15. | Haydite.......... | 75 | 36.1 | 15.6 | 9.0 | 3.4 | 480 | 670 | 670 |
| 5 BY 8 BY 12-IN. UNITS; 2 RECTANGULAR CORES; CORE AREA 57 PER CENT |  |  |  |  |  |  |  |  |  |
| No. 12. | Sand and Gravel.. | 122 | 19.6 | 7.8 | 3.9 | 2.8 | ... | 1420 | 1360 |
| No. 21. | Sand and Gravel.. | ... | .. | ... | ... | ... | ... | .... | 1480 |
| 3½ BY 8 BY 12-IN. UNITS; 2 RECTANGULAR CORES; CORE AREA 30 PER CENT |  |  |  |  |  |  |  |  |  |
| No. 13. | Sand and Gravel.. | 126 | 16.2 | 9.2 | 2.6 | 1.4 | ... | 1150 | 1550 |

The units were cured for 48 hours in a steam curing room, heated by steam coils along one side of the room and with moisture supplied from open petcocks on the lower steam coil. The temperature of the curing room was held at about 100° F. at night and at 80° F. in the daytime. Upon removal from the curing room, the units were stock-piled under cover until 21 days old, when they were delivered to the laboratory.

The cement content of the different lots was varied with the object of producing units in two strength ranges: one just passing specification requirements of 700 lb. per sq. in. at 28 days and the other having a strength of approximately 1100 to 1200 lb. per sq. in. Details of the mixtures used are given in Table II.

The 5 by 8 by 12-in. units (lot 12) donated by the Penniman Concrete and Material Co., Dallas, Tex., were their regular plant product. The same is true of the $3\frac{1}{2}$ by 8 by 12-in. units (lot 13) made by the Stone Tile Supply Co., Memphis, Tenn.

*Properties of Units.*—From each lot of block and tile used in the test walls, 20 units were used to determine physical properties. The tests of units were in general accordance with the specifications of the American Concrete Institute,[1] though compression tests were made on the units in their normal air-dry condition, to simulate the condition of the corresponding units in the wall. Units for compression test were capped with a mixture of plaster of Paris and cement one day prior to test. The absorption test was made on units after oven drying for 48 hours at 175° F., followed by 48-hour saturation. The unit weight was determined from the volume displacement of saturated blocks and the weight of oven-dry blocks. Data from the physical tests are given in Table III, values given being the average of 5 tests, except unit weights which are the average of 3 tests.

*Mortar.*—The proportions of the two mortar mixes used are based on dry, rodded volumes of sand. The sand was from Lincoln, Ill., and was graded to pass a No. 16 sieve. The $1:1:4\frac{1}{2}$ mix of cement, lime and sand by dry volume is not greatly different from the 1:1:6 mix frequently used with moist sand in construction work. The 1:3 cement mortar with 10 per cent of lime, by weight of cement, is representative of lean mortars and under field conditions of measurements would be denoted as about a $1:3\frac{3}{4}$ mix. The proportioning of materials for mortar was done by weight. The unit weights used were: cement, 94 lb.; lime, 40 lb.; and sand, 106 lb. From each batch, nine 2 by 4-in. control cylinders were made, cured for 48 hours in the molds and then stored with the walls. Three cylinders were tested in compression at the age of 7 days, three at 32 days when the walls were tested, and three were tested for absorption at 32 days. Results of the mortar tests are tabulated with the results of the wall tests.

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[1] Standard Specifications for Concrete Building Block and Concrete Building Tile (P-1-A-29), *Journal*, Am. Concrete Inst., No. 8, April, 1931, p. 1017.

*Brick.*—The physical properties of the face brick used in the composite walls (lots 16 and 17) when tested in accordance with the Tentative Methods of Testing Brick (A.S.T.M. Designation: C 67 – 30 T) were as follows: Compressive strength, 12,000 lb. per sq. in.; modulus of rupture, 1820 lb. per sq. in.; absorption, 7 per cent; dimensions, $2\frac{3}{16}$ by $3\frac{3}{4}$ by $8\frac{1}{4}$ in.

## FABRICATION OF WALLS

To provide a rigid base on which the walls could be built and transported, reinforced concrete pallets were used. They were $5\frac{3}{4}$ in. thick, of sufficient area to support the wall, and were poured on a machined surface so that no bedding was required when pallet and wall were placed in the testing machine. The pallet was stiff enough to prevent appreciable deflection during the fabrication of the wall. Preparatory to building a wall, the pallet was placed on two carefully levelled steel rails, and a frame carrying two vertical 2 by 2-in. angles 11 ft. long, forming guides for two corners of the wall, was placed against the pallet. This simplified laying the wall up plumb and normal to the plane of the base.

The walls were built by a union mason and helper, hired by the hour. They were instructed to follow their usual style of workmanship, except where different types of mortar and bedding were required by the schedule. Vertical joints were not filled, beyond buttering the edge of the unit as laid and final pointing of joints. In the brick courses of the composite walls, the mortar bed was spread, not furrowed. In these walls, in which backup blocks of Haydite or cinder concrete were used, each header brick was placed over a web of the concrete unit.

However, it was found after the walls were tested that through error on the part of the mason only a small percentage of the webs of Haydite units directly above the header courses were effectively bedded, while the webs of cinder units were fully bedded. This unintentional difference in workmanship makes impossible a comparison between the walls of lots 16 and 17, since in lot 16 the units have virtually a face-shell mortar bed, and in lot 17 a full mortar bed was used.

In general the main group of walls probably represent average building construction in this locality.

The walls were built from units that were 28 days old, having received plant curing for 21 days and dry storage in the laboratory for 7 days. The mortar was mixed by hand by the mason's helper who used fixed proportions of dry materials but varied the mixing water to produce the consistency desired. The amounts of water used were recorded. The unit weight of the mortar and the amount required in each wall were noted.

The walls were air-cured in the laboratory, at an average temperature of about 73° F., and a relative humidity of 50 to 65 per cent, for 32 days before testing. Figure 1 shows a number of the walls in storage. They were usually capped one day before test with a layer of neat, high-early-strength cement. The steel capping plate, 2 in. thick, was left in place and used as a bearing plate during the test. The walls were moved to the testing machine by means of a 10-ton traveling crane. Two $\frac{7}{8}$-in. rods, passing through pipe sleeves in the pallet, provided means of attaching lifting straps suspended from the crane and also served for the attachment of casters used in moving the wall into the testing machine.

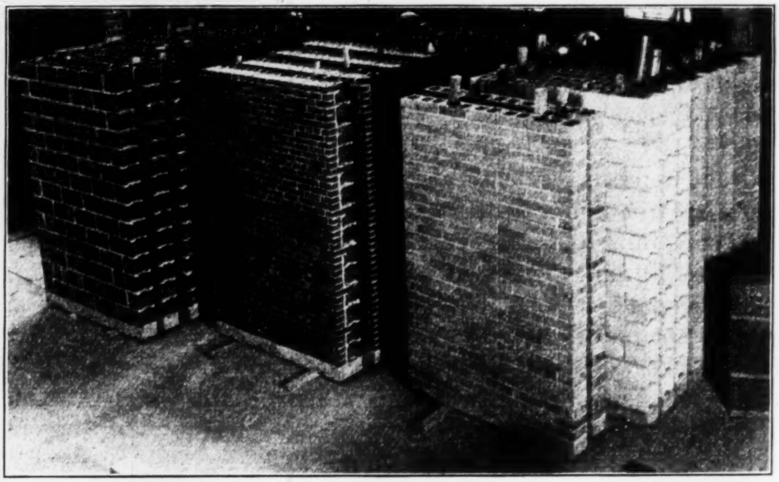

FIG. 1.—View of Walls in Storage Before Testing.

## TESTING PROCEDURE

The walls of lots 1 to 17, inclusive, were tested in axial compression. The large walls were tested 32 days after erection, in either a Southwark-Emery machine of 3,000,000-lb. capacity or in a Riehle machine of 600,000-lb. capacity, with a test speed of about 0.05 in.

per minute. Due to care in the preparation of pallets and laying up of walls, the alignment of specimens in the machine was very good. Load was applied through a spherical bearing block and two 24-in. I-beams to the heavy steel bearing plate used to cap the wall. The stiffness of the I-beams was sufficient to distribute the load very uniformly over the length of the wall. To measure axial strains, four vertical compressometers of 100-in. gage length were used, two on each side of the wall 18 in. from the center line. A vertical wire, 100 in. long, stretched opposite a mirror and scale, permitted the observation of lateral deflection or buckling of the wall, with a precision of 0.01 in. On one wall of each lot, strain-gage measurements were also taken, on 2-in. gage lines across mortar joints and on 2- or 6-in. gage lines on the units. These readings were taken with Berry strain gages on steel plugs set in the wall near mid-height. Figure 2 shows a wall in the Southwark-Emery testing machine, after being tested to failure.

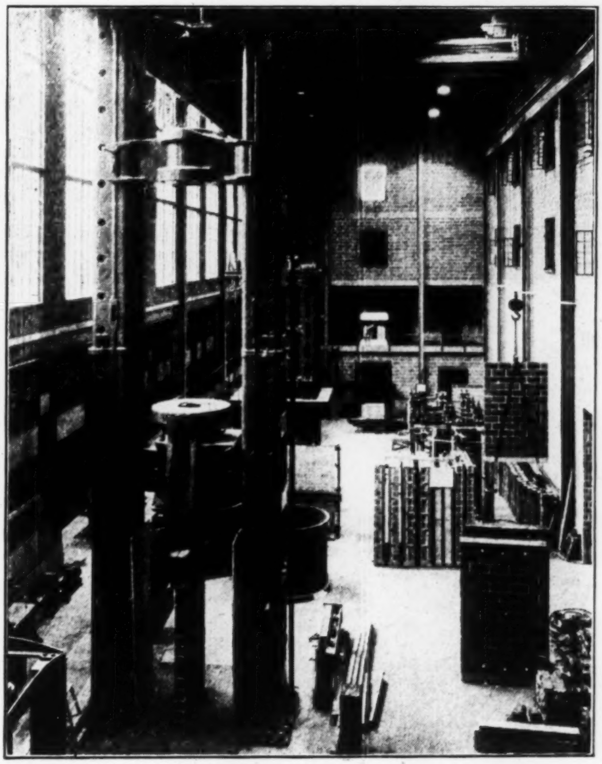

FIG. 2.—Wall in Southwark-Emery Testing Machine.

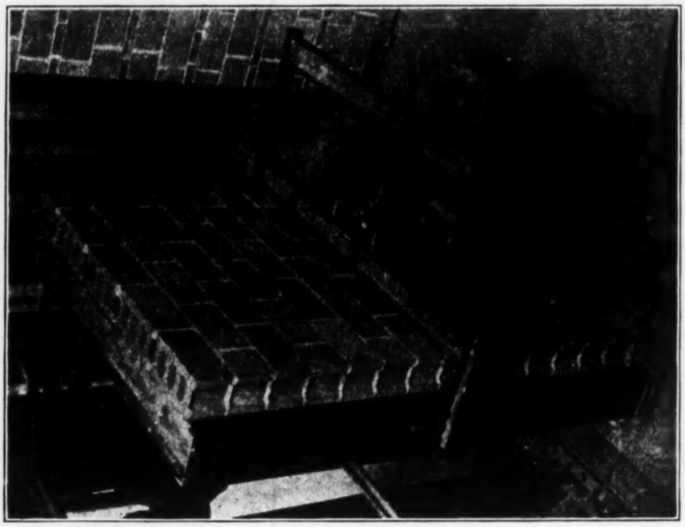

FIG. 3.—Wall in Flexure Testing Apparatus.

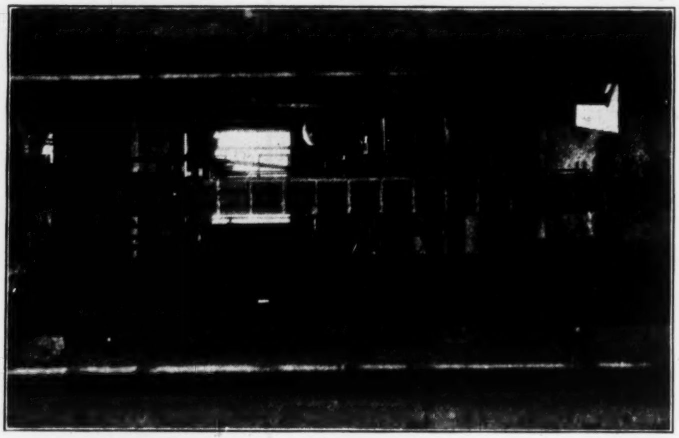

FIG. 4.—Wall Under Eccentric Loading.

TABLE IV.—PRINCIPAL RESULTS OF MASONRY WALL TESTS.

Large Walls—6 ft. long, 9 ft. 6 in. high, tested in compression and flexure.
Small Walls—2 ft. 8 in. long, 4 ft. high, tested in compression.
All walls built when units were 28 days old, stored in air of laboratory and tested 32 days later.
Strengths of walls and units are based on gross areas.
Compressive strength of units given are from 60-day tests.
Compressive strength of mortars from 2 by 4 in. cylinders, 32 days old.
All units manufactured in products plants under usual conditions.
Mortar joints, full mortar bed except walls marked 1CS, 2HS, 3GS and 21GS, which were bedded on face shells only.
Composite walls, 4-in. face brick, 8-in. concrete back-up unit.
Eccentric loads applied at edge of middle third of wall thickness.
The unit stresses given for eccentrically loaded walls are the average unit stresses.

| Wall | Type of Unit | Aggregate | Mortar Mix by Volume, Dry Rodded | Maximum Load on Walls | | | | Compressive Strength, lb. per sq. in. | | | Modulus of Rupture of Large Wall, lb. per sq. in. | Ratio of Strength of Large Wall to Strength of | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Large | | Small | | Units | Mortar | | | | |
| | | | | lb. | lb. per sq. in. | lb. | lb. per sq. in. | | Large | Small | | Small Wall | Unit |
| | | | | AXIAL LOADING | | | | | | | | | |
| 1CF 1 | 8 by 8 by 16 in. 3-oval-core | Cinder | 1:1:4½ | 246 000 | 430 | 118 000 | 470 | | 1180 | 1180 | | | |
| 2 | | | | 250 000 | 435 | 107 700 | 430 | | 1440 | 1180 | 34 | | |
| 3 | | | | 268 000 | 465 | | | | 1440 | | | | |
| | | | Average | 254 700 | 445 | 112 850 | 450 | 865 | | | | 0.99 | 0.51 |
| 1CS 1 | 8 by 8 by 16 in 3-oval-core | Cinder | 1:1:4½ | 193 000 | 335 | 105 550 | 420 | | 1060 | 1160 | | | |
| 2 | | | | 204 000 | 355 | 88 550 | 350 | | 900 | 900 | 42 | | |
| 3 | | | | 204 000 | 355 | | | | 1920 | | | | |
| | | | Average | 200 300 | 350 | 97 050 | 380 | 865 | | | | 0.92 | 0.40 |
| 2HF 1 | 8 by 8 by 16 in. 3-oval-core | Haydite | 1:1:4½ | 267 000 | 465 | 136 600 | 540 | | 1230 | 1230 | 47 | | |
| 2 | | | | 246 000 | 430 | 99 400 | 395 | | 730 | 730 | | | |
| 3 | | | | 271 000 | 470 | | | | 1130 | | | | |
| | | | Average | 261 300 | 455 | 118 000 | 470 | 820 | | | | 0.97 | 0.56 |
| 2HS 1 | 8 by 8 by 16 in. 3-oval-core | Haydite | 1:1:4½ | 208 000 | 360 | 91 950 | 365 | | 960 | 960 | | | |
| 2 | | | | 218 000 | 380 | 103 000 | 410 | | 780 | 780 | 28 | | |
| 3 | | | | 217 000 | 375 | | | | 1250 | | | | |
| | | | Average | 214 300 | 370 | 97 475 | 390 | 820 | | | | 0.95 | 0.45 |
| 3GF 1 | 8 by 8 by 16 in. 3-oval-core | Sand and Gravel | 1:1:4½ | 328 000 | 570 | 177 000 | 700 | | 880 | 880 | | | |
| 2 | | | | 386 000 | 670 | 153 800 | 610 | | 750 | 810 | 19 | | |
| 3 | | | | 336 000 | 585 | | | | 810 | | | | |
| | | | Average | 350 000 | 610 | 165 400 | 655 | 1070 | | | | 0.93 | 0.57 |
| 3GS 1 | 8 by 8 by 16 in. 3-oval-core | Sand and Gravel | 1:1:4½ | 276 000 | 480 | 123 100 | 490 | | 670 | 810 | | | |
| 2 | | | | 257 100 | 450 | 126 100 | 500 | | 1020 | 670 | 23 | | |
| 3 | | | | 240 000 | 420 | | | | 1020 | | | | |
| | | | Average | 257 700 | 450 | 124 600 | 495 | 1070 | | | | 0.91 | 0.42 |
| 4LF 1 | 8 by 8 by 16 in. 3-oval-core | Limestone | 1:1:4½ | 211 000 | 365 | 127 400 | 505 | | 1120 | 1080 | | | |
| 2 | | | | 230 800 | 400 | 105 500 | 420 | | 1150 | 1080 | | | |
| 3 | | | | 230 000 | 400 | | | | 1080 | | 18 | | |
| | | | Average | 223 900 | 390 | 116 450 | 460 | 780 | | | | 0.85 | 0.50 |
| 5SF 1 | 8 by 8 by 16 in. 3-oval-core | Slag | 1:1:4½ | 316 100 | 550 | 148 900 | 590 | | 1270 | 1270 | | | |
| 2 | | | | 319 300 | 555 | 149 250 | 590 | | 940 | 1270 | 27 | | |
| 3 | | | | 296 500 | 515 | | | | 1120 | | | | |
| | | | Average | 310 600 | 540 | 149 075 | 590 | 1290 | | | | 0.92 | 0.42 |
| 6HF 1 | 8 by 8 by 16 in. 3-oval-core | Haydite | 1:1:4½ | 254 600 | 445 | 131 150 | 520 | | 1380 | 1670 | 18 | | |
| 2 | | | | 269 400 | 470 | 129 850 | 515 | | 1380 | 1670 | | | |
| 3 | | | | 233 700 | 405 | | | | 1670 | | | | |
| | | | Average | 252 600 | 440 | 130 500 | 520 | 1280 | | | | 0.85 | 0.34 |
| 7CF 1 | 8 by 8 by 16 in. 3-oval-core | Cinder | 1:1:4½ | 302 000 | 525 | 161 700 | 640 | | 1920 | 980 | 18 | | |
| 2 | | | | 291 000 | 505 | 139 200 | 550 | | 980 | 980 | | | |
| 3 | | | | 297 000 | 515 | | | | 1280 | | | | |
| | | | Average | 296 700 | 515 | 150 450 | 595 | 970 | | | | 0.87 | 0.53 |
| 8GF 1 | 8 by 8 by 16 in. 3-oval-core | Sand and Gravel | 1:1:4½ | 402 000 | 700 | 170 000 | 675 | | 800 | 880 | 38 | | |
| 2 | | | | 407 000 | 710 | 148 900 | 590 | | 800 | 880 | | | |
| 3 | | | | 391 000 | 680 | | | | 750 | | | | |
| | | | Average | 400 000 | 695 | 159 450 | 635 | 1230 | | | | 1.09 | 0.57 |

TABLE IV.—*Continued.*

| Wall | Type of Unit | Aggregate | Mortar Mix by Volume, Dry Rodded | Maximum Load on Walls | | | | Compressive Strength, lb. per sq. in. | | | Modulus of Rupture of Large Wall, lb. per sq. in. | Ratio of Strength of Large Wall to Strength of | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Large | | Small | | Units | Mortar | | | Small Wall | Unit |
| | | | | lb. | lb. per sq. in. | lb. | lb. per sq. in. | | Large | Small | | | |
| AXIAL LOADING | | | | | | | | | | | | | |
| 9CF 1 | 8 by 8 by 16 in. 3-oval-core | Cinder | 1:3+10 per cent Lime | 350 100 | 610 | 158 700 | 630 | | 1730 | 1730 | | | |
| 2 | | | | 347 500 | 605 | 159 800 | 635 | | 2200 | 1730 | 22 | | |
| | | | Average | 348 800 | 610 | 159 250 | 635 | 1150 | | | | 0.96 | 0.53 |
| 10GF 1 | 8 by 8 by 16 in. 3-oval-core | Sand and Gravel | 1:3+10 per cent Lime | 220 100 | 380 | 98 700 | 390 | | 1770 | 1770 | | | |
| 2 | | | | 189 000 | 330 | 86 700 | 345 | | 1860 | 1770 | 23 | | |
| | | | Average | 204 550 | 355 | 92 700 | 370 | 550 | | | | 0.96 | 0.65 |
| 11GF 1 | 8 by 8 by 16 in. 3-oval-core | Sand and Gravel | 1:3+10 per cent Lime | 450 000 | 780 | 230 000 | 915 | | 2100 | 1870 | | | |
| 2 | | | | [a] | ... | 242 700 | 965 | | 1870 | 1870 | 36 | | |
| | | | Average | 450 000 | 780 | 236 350 | 940 | 1570 | | | | 0.83 | 0.50 |
| 12GF 1 | 5 by 8 by 12 in. 2-square-core | Sand and Gravel | 1:1:4½ | 324 500 | 590 | 153 200 | 670 | | 1230 | 1230 | 46 | | |
| 2 | | | | 317 000 | 580 | 169 650 | 740 | | 1270 | 1270 | | | |
| | | | Average | 320 750 | 585 | 161 425 | 705 | 1360 | | | | 0.83 | 0.43 |
| 13GF 1 | 3½ by 8 by 12 in. 2-square-core | Sand and Gravel | 1:1:4½ | 320 000 | 565 | 173 500 | 700 | | 1180 | 1140 | | | |
| 2 | | | | 390 000 | 690 | 212 000 | 855 | | 1300 | 1140 | 28 | | |
| | | | Average | 355 000 | 630 | 192 750 | 775 | 1550 | | | | 0.81 | 0.41 |
| 14GF 1 | 8 by 12 by 16 in. 3-oval-core | Sand and Gravel | 1:1:4½ | 298 000 | 345 | 155 900 | 410 | | 1130 | 1130 | | | |
| 2 | | | | 337 000 | 390 | 148 800 | 390 | | 1010 | 1130 | 18 | | |
| | | | Average | 317 500 | 370 | 152 350 | 400 | 600 | | | | 0.93 | 0.62 |
| 15HF 1 | 8 by 12 by 16 in. 3-oval-core | Haydite | 1:1:4½ | 320 000 | 370 | 136 000 | 360 | | 1070 | 1070 | 27 | | |
| 2 | | | | 308 000 | 355 | 140 600 | 375 | | 840 | 1070 | | | |
| | | | Average | 314 000 | 365 | 139 300 | 370 | 670 | | | | 0.99 | 0.54 |
| 16HS 1 | 8 by 8 by 16 in. 3-oval-core | Haydite and Brick | 1:1¼:4½ | 446 000 | 495 | 237 000 | 585 | | 1260 | 1020 | | | |
| 2 | | | | 532 000 | 595 | 279 800 | 690 | | 1900 | 1020 | 38 | | |
| | | | Average | (489 000)[d] | (545)[d] | (258 400)[d] | (640)[d] | (520)[c]<br>720 | | | | 0.85 | (1.05)[c]<br>(0.76)[d] |
| 17CF 1 | 8 by 8 by 16 in. 3-oval-core | Cinder and Brick | 1:1¼:4½ | 760 000 | 850 | 318 000 | 790 | | 1045 | 1050 | | | |
| 2 | | | | 677 000 | 755 | 309 700 | 770 | | 1305 | 1050 | 50 | | |
| | | | Average | 718 500 | 805 | 313 850 | 780 | (1050)[c]<br>810 | | | | 1.03 | (0.77)[c]<br>0.99 |
| ECCENTRIC LOADING | | | | | | | | | | | | | |
| 18HF 1 | 8 by 8 by 16 in. 3-oval-core | Haydite | 1:1:4½ | 217 000 | 375 | 110 600 | 440 | | 990 | 990 | .. | | |
| 2 | | | | 235 000 | 410 | 129 000 | 510 | | 900 | 900 | | | |
| 3 | | | | 235 000 | 410 | | | | 960 | | | | |
| | | | Average | 229 000 | 400 | 119 800 | 475 | 1040 | | | | 0.84 | 0.38 |
| 19CF 1 | 8 by 8 by 16 in. 3-oval-core | Cinder | 1:1:4½ | 209 800 | 365 | ...... | ... | | 1160 | ... | .. | | |
| 2 | | | | 199 000 | 345 | ...... | ... | | 1450 | ... | | | |
| | | | Average | 204 400 | 355 | | | 850 | | | | .... | 0.42 |
| 20GF 1 | 8 by 8 by 16 in. 3-oval-core | Sand and Gravel | 1:1:4½ | 248 700 | 430 | ...... | ... | | 1580 | ... | .. | | |
| 2 | | | | 240 500 | 420 | ...... | ... | | 1580 | ... | | | |
| | | | Average | 244 600 | 425 | | | 1010 | | | | .... | 0.42 |
| 21GS 1[b] | 5 by 8 by 12 in. 2-square-core | Sand and Gravel | 1:1:4½ | 299 000 | 545 | ...... | ... | | 1190 | ... | .. | | |
| 2 | | | | 270 000 | 490 | ...... | ... | | 1370 | ... | | | |
| | | | Average | 284 500 | 515 | | | 1480 | | | | .... | 0.35 |

[a] Broken while placing in testing machine.

[b] Units 3 months old when wall was built. Walls tested 32 days later.

[c] Strengths of offset blocks used in alternate courses of composite walls.

[d] These walls laid up, by error, with face shell bedding. The results are not comparable with those of Lot 17, in which ul l bedding was used.

The small walls were tested in the same manner as the large ones except that vertical deformations were measured by a 40-in. compressometer at the middle of each side of the wall.

A flexure test was made on one of the large walls of each lot. These walls, which failed by cracking along a horizontal joint near mid-height, were later tested in compression, and developed as good compressive strengths as the uncracked walls. The flexure test was made by placing the wall against a vertical structural steel framework and applying a lateral concentrated load along the horizontal center line, with the slab supported along the top and bottom edges. The span was 9 ft. The load was applied by means of a 10-ton screw jack, a spring dynamometer and a steel I-beam, as shown in Fig. 3. All points of bearing against the wall were cushioned with a strip of 3-in. leather belting. The dynamometer, in which deflections were measured by an Ames dial graduated to 0.001 in., was calibrated in a 10,000-lb. Olsen testing machine. Deflections of the wall were read to a precision of 0.01 in.

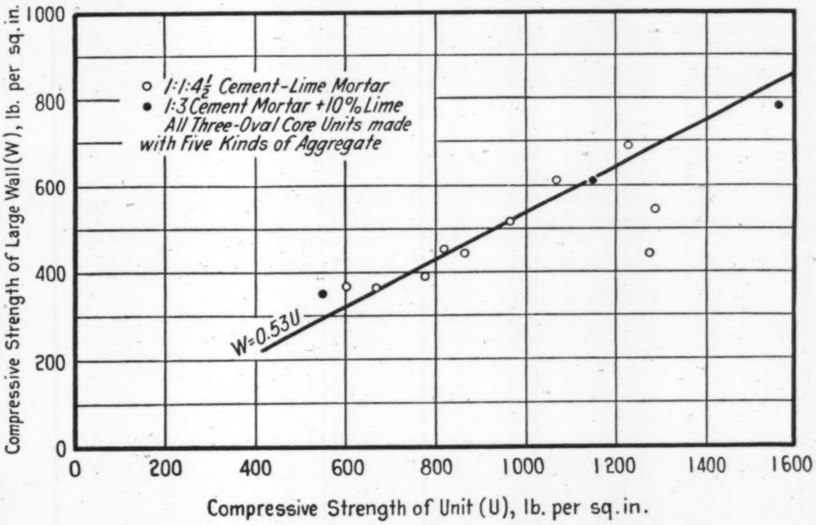

FIG. 5.—Relation Between Strengths of Walls and of Units.

Eccentric compressive loads were applied to the walls of lots 18 to 21. The procedure differed from the ordinary compression test only in that load was applied at the top of the wall through a ¾-in. square cold-rolled steel rod resting on the top bearing plate and one-sixth of the wall thickness from the central plane of the wall. The arrangement is shown in Fig. 4. No form of eccentric support was used at the bottom of the wall. The strains and deflections were measured as in axial load tests, with some additional strain readings at the top and bottom of the wall to indicate stress distribution.

## DISCUSSION OF RESULTS

The principal results of the wall tests are given in Table IV. It is of interest that the compressive strengths of all large walls including all grades of units, composite construction and even the effect of eccentric loading, fell between 335 and 850 lb. per sq. in. of gross section. A study of the tabulated data shows that a fairly definite answer has been found to the questions studied. The major results are summarized in the following paragraphs.

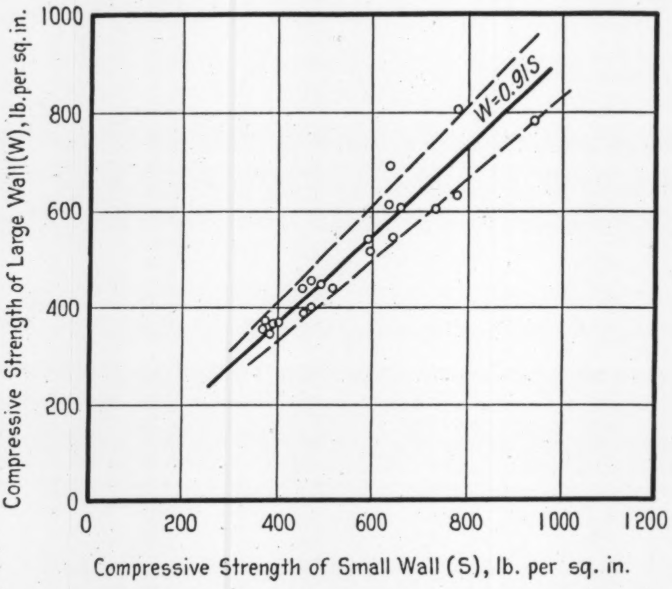

FIG. 6.—Relation Between Strengths of Large and Small Walls.

*Effect of Strength of Unit on Wall Strength.*—It appears from inspection of Table IV that the wall strengths are principally dependent on the strength of the building unit used. The last column of the table shows that the ratio of strength of wall to that of unit, for large walls made with 3 oval core units and full mortar bed, is fairly constant with an average value of 0.53. While the ratio varies from 0.34 to 0.65 for the 13 walls, the values for nine of these lots lie between 0.50 and 0.57. The consistency of the test results is best seen by reference to Fig. 5, in which wall strength is plotted against strength of unit. The average relation between the two is shown by the solid curve, and the variation from this curve is seen to be relatively small.

*Relation Between Strengths of Large and Small Walls.*—The compressive strengths of all large and small walls, including the composite and eccentrically loaded panels, are plotted in Fig. 6. The plotted points lie within a fairly narrow zone. The average ratio of the strength of large walls to that of wallettes is 0.91 and this relation is represented by the solid curve in the figure. The two broken lines, indicating roughly the limits of the zone, correspond to strength ratios of 0.82 and 1.00. The tests of composite walls, lots 16 and 17, furnish two of the extreme variations from the average curve. The wallettes gave quite consistent results and the indications are that wallette strengths, multiplied by a factor of 0.9, might well be considered as representative of the strength of full-story panels, such as the large panels tested.

*Effect of Type of Aggregate.*—The wall tests do not show marked differences in the effectiveness of units made with different aggregates. Due to differences in the strengths of units, comparisons must of necessity be made by considering the ratio of wall strength to unit strength, as given in the last column of Table IV. Since experimental variations within one lot are frequently as great as the variations between lots containing different aggregates, definite comparisons are difficult. An aggregate may be associated with a high ratio in one group and a low one in another. In lots 1 to 4, with a range in 28-day strength of units from 730 to 830 lb. per sq. in., the total range in wall strengths was about 8 per cent, or 65 lb. per sq. in. on an 800-lb. unit.

Comparison of the stronger units, lots 5 to 8, shows a rather wide range in the strength ratio of walls to units, since lots 5 and 6 gave the lowest values found. A study of lots 2 and 6, both made with Haydite aggregates, shows a greater wall strength with the 820-lb. units of lot 2 than with the 1280-lb. units of lot 6. This would not seem attributable to the kind of aggregate used. It may be due to the fact that the units of lot 6 had a very high moisture content when laid in the walls and that they dried out less as stored in the walls than did the corresponding individual units tested. A similar argument applies to the walls of lot 5.

*Effect of Type of Mortar.*—In a number of wall tests, signs of incipient failure were noted at the junction of mortar joint and unit, though no marked differences in the efficiency of units due to differences in mortar strength were found. Tests were outlined to furnish a comparison of mortar, lots 1, 3 and 8 made with full bed of $1:1:4\frac{1}{2}$ cement-lime mortar being paralleled by lots 9, 10 and 11 made with 1:3 cement mortar containing 10 per cent lime. The respective values of the wall-unit strength ratio are 0.51, 0.57 and 0.57 for the first three and 0.53, 0.65 and 0.50 for the second three. The averages for the two groups are practically alike and there is no consistent or systematic variation. This is not especially strange, since it is evident that the mortar strengths were considerably above the wall strengths developed, even when net bearing sections are considered.

*Effect of Kind of Mortar Bed.*—In planning the tests it was decided to lay the units in a full mortar bed, as representative of good supervised construction. At the same time, in recognition of the fact that in some localities it is common to bed only the outer face shells of the units, the walls of lots 1, 2 and 3 were laid up in both ways, thus affording a comparison of results from 9 large walls and 6 wallettes made with each style of mortar bed. The results are quite consistent for the three lots and the two sizes of wall. The ratio of wall strengths with face shell bedding to those with full bedding is as follows: Large walls: lot 1, 0.79; lot 2, 0.81; lot 3, 0.74; wallettes: lot 1, 0.84; lot 2, 0.83; lot 3, 0.76. The average for the six values is 0.80. For the oval-core units to which these data apply, the cores are $4\frac{7}{8}$ in. wide, leaving the face shells with a minimum thickness at the core of $1\frac{1}{2}$ in. Considering that the width of the mortar bed on the face shells might average 2 in., the ratio of the bedded area to the net area of the unit is very nearly 0.8. Hence it may be said that for the 8 by 8 by 16-in. units the ratios of wall strengths are substantially proportional to the bearing areas of the mortar beds, and that this ratio may be taken at about 0.8. It seems reasonable that this factor might be applied to estimate the strength of other walls, such as those of lots 4 to 11, assuming that they were to be laid with face shell bedding.

*Effect of Design of Unit.*—Three styles of units were used in the 8-in. walls, the 8 by 8 by 16-in. block, with 3 oval cores, the 5 by 8 by 12-in. tile and the $3\frac{1}{2}$ by 8 by 12-in. tile, with 2 rectangular cores. The walls of lots 8, 12 and 13 were made with these three kinds of units and gave wall strengths of 695, 585 and 630 lb. per sq. in., respectively, on units within a comparable range of strength. The respective values of the ratio of large wall strength to strength of unit are 0.57, 0.43 and 0.41; similar ratios between strengths of wallettes and units are 0.52, 0.52 and 0.50. The average for both sizes of wall are 0.55, 0.47 and 0.46. An explanation of the lower ratios for the tile walls may be that the middle web of a tile unit is placed above the vertical joint between end webs of units in the course below, or *vice versa,* and since these walls were laid with a full mortar bed, the percentage of the total net area embedded was undoubtedly somewhat lower than that obtained with the 3-oval-core units.

The values quoted above indicate that the mortar bed was about 85 per cent effective for the tile units, which is somewhat greater than the value of 0.80 for relative effectiveness of block with face shell bedding. Since the tile units are customarily laid with face shell bedding only, the lack of continuity of webs is probably of much less importance than is indicated by the tests of lots 12 and 13.

*Effect of Wall Thickness.*—The 8 and 12-in. walls of lots 2, 3, 14 and 15 were intended for comparison, since they were made of blocks similar except in thickness. Actually, the units of lots 2 and 3 were considerably stronger than the others. Comparing the ratios of strength of wall to that of unit (including both large and small walls) the following values are found: for 8-in. walls, lot 2, Haydite, 0.56; lot 3, sand-gravel, 0.59; for 12-in. walls, lot 15, Haydite, 0.54; lot 14, sand-gravel, 0.64. The average value for the 8-in. walls is thus 0.58; that for 12-in. walls, 0.59. From these tests of walls with full bedding, there would seem to be no difference in the efficiency of walls as based upon the strength of the units.

*Composite Walls.*—In the walls of lots 16 and 17, Haydite and cinder back-up blocks were combined with a 4-in. thickness of face brick to form composite panels, 12¼ in. thick. The walls were tested to study the effectiveness of action of the two types of material in the same wall, with particular attention to the deformations of the two materials. The failure of all of these walls was characterized by an initial yielding and apparent readjustment of loads, accompanied by some cracking and splitting of units, at a load considerably below the maximum. One of the walls made with Haydite units, after exhibiting vertical cracks through the back-up units, and a decrease in load after reaching 462,000 lb., continued to offer resistance with further loading and finally failed at a load of 532,000 lb. In general, in these composite walls, the lateral bowing or deflection of the walls was marked and at failure the wall split either along a plane crossing the webs of the backup units, or along the joint between face brick and concrete sections. The ratio of overall wall strength to the strength of the back-up units was considerably greater than the corresponding ratio for plain concrete masonry walls. The eccentricity of the resistance of the wall was evidently more than compensated for by the resistance of the solid brick masonry. Average values of the measured strains indicate that the relative deformations at early loads on the brick and concrete wall faces were, for brick and Haydite walls, 1.0 and 1.4, and for brick and cinder block walls, 1.0 and 2.6.

Since the walls of lot 17 were made with full mortar beds and those of lot 16 were laid up, by error, with face shell bedding, direct comparison of the results is impossible. The face shell bedding used on lot 16 would be expected to precipitate the splitting failure through the webs of the back-up units, which was observed in the tests. The difference in strength noted in Table IV between the offset and regular units used in alternate wall courses also confuses the relation between strength of wall and strength of units. It is planned to make further tests of walls of this type in which identical conditions of workmanship and type of mortar bed may be secured.

*Flexural Strength of Walls.*—Values of the modulus of rupture of a large wall of each lot of units are given in Table IV. These values are computed on the basis of a beam of solid rectangular section, with a width equal to the length of the wall and a depth equal to the wall thickness. This assumption is used to facilitate computation and gives values that err on the side of conservatism. The failure of the walls was in all cases a failure in adhesion of mortar to unit. In all of the oval-core units, molded on a metal pallet, the failure was between the smooth, molded lower surface of the block and the mortar. There does not seem to be any definite relation between the modulus of rupture and the compressive strength of mortar cylinders. The walls with face-shell bedding give about as good results, considering the mortar strengths, as do corresponding walls with full bedding. This is logical, since the area occupied by cross webs should have little effect on flexural resistance.

The values of modulus of rupture found vary from 18 to 50 lb. per sq. in. The low values were found with mortars of various compressive strengths. The composite walls, tested with the brick faces in tension, gave values of 38 and 50 lb. per sq. in., which are among the highest of the group. It appears that the modulus of rupture of concrete masonry walls depends principally upon the texture and absorptive properties of the surface of the units at the mortar joints. Neither units nor bricks were wetted or otherwise treated before being laid up in the wall. If it seemed desirable to secure increased flexural strength of walls, it might be accomplished by securing a rougher texture of the bearing faces or by wetting the faces to produce an optimum "suction" between unit and mortar.

*Eccentrically Loaded Walls.*—The results of the tests of eccentrically loaded walls of lots 18, 19, 20 and 21, may be compared with those for axially loaded walls of lots 1, 2, 3 and 12, respectively. The comparison between lots 21 and 12 is disturbed, however, by the fact that the walls of lot 21 were laid up with face shell bedding, while those of lot 12 were laid up with full mortar beds. As explained in the section on the effect of design of units, this difference in type of bedding should have slight effect with this type of unit, in which a complete bedding of cross webs is prevented by their spacing.

The load was applied to the walls with an eccentricity equal to one-sixth of the wall thickness. In a homogeneous, rectangular wall, this should produce a stress distribution varying from zero at one face to a maximum (twice the average stress) at the other face. If the material failed at a limiting maximum fiber stress, the eccentrically loaded wall should carry only one-half the total load carried by an axially loaded wall. It is known, however, that the compressive strength of concrete in flexure is greater than the strength in axial compression. Moreover, the hollow units provide a condition more favorable to eccentric loading than does a solid rectangular section.

The ultimate loads on eccentrically loaded walls are given in Table IV, together with the *average* stress, and in the last column this stress is expressed as a ratio to the strength of the units, just as for axially loaded walls. The ratios for the four lots, 0.38, 0.42, 0.42 and 0.35, may be compared with the values for lots 2, 1, 3 and 12, which are 0.51, 0.56, 0.57 and 0.43. The walls under eccentric loading are thus seen to be, respectively, 74, 75, 74 and 81 per cent as strong as similar walls axially loaded. While these tests show that the load-carrying ability of concrete masonry walls is even better than might be expected from theoretical considerations, it also shows that a fairly small eccentricity of load produces a very definite and consistently exhibited reduction in load-carrying capacity.

## CONCLUSIONS

The principal conclusions from the investigation may be summarized as follows:

1. The compressive strengths of the large walls of all types varied from 335 to 850 lb. per sq. in.

2. The strength of walls was evidently dependent mainly upon the strength of the building units and was little affected by variations in strength of mortar.

3. The ratio of the compressive strength of large walls to the 60-day compressive strength of units used was found to be quite well defined. The average value of the ratio is 0.53.

4. The ratio between the compressive strength of large walls and of wallettes was also fairly constant, with an average value of 0.91. This result suggests the feasibility of using wallettes as representative of wall construction in further tests.

5. Tests of walls made of 8-in. units with 3 oval cores showed that the compressive strength with face-shell mortar beds was 80 per cent of that produced by the use of full mortar beds.

6. Little definite effect could be established as due to variations in type of aggregate, type of mortar or thickness of walls. Some differences found with three types of unit could be explained by the arrangement of cross-webs of the units.

7. Composite walls of face brick and concrete building units showed good inter-action of the two types of unit, and quite satisfactory strengths, even though the walls developed a marked deformation and deflection before failure.

8. The flexural strength of the walls was evidently a function of the adhesion of the mortar to the bearing surface of the unit, since all transverse failures occurred at the junction of the two materials. Values of modulus of rupture for the walls vary from 18 to 50 lb. per sq. in.

9. The walls in which the eccentric compressive load was applied at the edge of the middle third of the wall thickness showed strengths averaging 76 per cent as great as were obtained with axial loading.

# DISCUSSION ON MASONRY BUILDING UNITS

Mr. A. R. Small[1] (*presented in written form*).—Mr. Parsons' paper includes, and quite sufficiently too, reasons why certain physical properties of hollow masonry units may not be taken alone as bases for allowable working loads upon bearing walls, etc., assembled of them. Economic factors will usually be controlling ones and properly so. Probably Mr. Parsons intended that workmanship should be classed as one economic factor.

There is a persisting inconsistency in the generally recognized allowable working loads on solid walls of common brick, our most generally recognized, if not oldest, form of loaded masonry construction. For example, the ratios for various grades of common brick, as given in Appendix I of the 1931 edition of the Building Code of the National Board of Fire Underwriters, are 20, 18, 14+ and 12. The last paragraph of the text of this Appendix states:

> Considering these factors and that the strength of the brick is influenced by the straightness and regularity of shape, and the type of surface of the brick, the thickness of the mortar joints and the rate and degree of absorption of the individual bricks, the working stresses fixed for brickwork should not be taken at more than one-fourth of the average ultimate compressive strength developed for a given set of conditions.

This text further explains: "When, however, the strength of the mortar joint has been reached, an increase in the strength of the brick does not add materially to the strength of the brickwork."

For walls of hollow units, the National Board of Fire Underwriter's code and those of various cities limit the allowable load to 80 lb. per sq. in. for the gross area without detailed consideration of the average strength of the units employed or their attitude (end or side construction) in the assembly. The ratio (21 ±) of this allowable working stress in the case of end-construction tile is of the approximate order of those for brick already mentioned, on the basis of the net cross-sectional area of the unit. Hence again, for end-construction tile, the limit of the strength of the mortar appears to be a controlling factor.

Oftentimes performance limits for one or more properties of a material are established and continued in order to secure a quality control for the product and, by indirection only, to indicate a relationship of the values for these properties to the appropriate uses of the materials. A notable example will be found in the Society's Tentative Specifications for Cotton Rubber-Lined Fire Hose (D 296 - 31T). The values of 1200 lb. per sq. in. for tensile strength, a stretch between 2-in. marks of at least 10 in. and a permanent set of 25 to 30 per cent are of major importance in determining the quality of the rubber lining and its correct vulcanization. They do not otherwise or directly reflect the loads or extensions to which the rubber parts of fire hose are subject when the hose is in use.

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[1] Vice-President, Underwriters' Laboratories, New York City.

Similarly it may be taken that the minimum values for strength in compression for the various forms of masonry units (for hollow units anyway) register controls of workmanship and other qualities in their manufacture and performance which are essential to the promoted uses and that the compressive strength of the units is not, by itself, the scale by which safe loads on masonry assemblies can be measured.

MR. H. F. GONNERMAN[1] (*presented in written form*).—Mr. Parsons has pointed out several factors, other than strength of unit, which have a bearing on the strength of hollow masonry walls, special emphasis being given to the design of the unit as it relates to the area of the mortar bedding. He has shown that this important factor should receive consideration in drafting specifications for masonry units.

Obviously specifications for strength and design of units should bear some relation to the load permitted on the finished wall. The maximum design load allowed on hollow masonry by the U. S. Department of Commerce publication, "Recommended Minimum Requirements for Masonry Wall Construction" (1924), and by numerous building codes is 70 lb. per sq. in. with a cement-lime mortar and 80 lb. per sq. in. with a portland-cement mortar. Based on these values and a reasonable factor of safety of 4, wall strengths of 280 lb. per sq. in. with a cement-lime mortar and 320 lb. per sq. in. with a cement mortar are established.

Mr. Parsons has developed a formula, based on tests of clay tile walls, which he has shown will serve as a basis for determining certain design limitations which may be found desirable to include in specifications or in building codes.

Except for tests conducted by the Underwriters' Laboratories on strength and stability of walls after fire exposure, there were few data available on the strength of story-height concrete masonry assemblies prior to the investigation of the stability of concrete masonry walls recently completed by F. E. Richart at the University of Illinois and presented in the paper by F. E. Richart, P. M. Woodworth and R. B. B. Moorman. Results from the University of Illinois' investigation were not available when Mr. Parsons' paper was written. Consequently, it is of interest to apply his formula to these data on concrete masonry.

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[1] Manager, Research Laboratory, Portland Cement Assn., Chicago, Ill.

TABLE I.—COMPRESSIVE STRENGTH OF CONCRETE MASONRY WALLS.[a]

| Reference Number | Authors'[a] Lot Number | Size and Type of Unit | Aggregate | Mortar | | Compressive Strength lb. per sq. in. | | Estimated Ratio of Bearing Area to Gross Area | Ratio of Wall Strength to Strength of Unit | Factor[b] of Safety |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Mix by Volume, Dry Rodded | Compressive Strength of 2 by 4-in. Cylinders, lb. per sq. in. | Masonry Units | Large Walls | | | |
| 1 | 1F..... | 8 by 8 by 16 in.<br>3-oval-core | Cinders | $1:1:4\frac{1}{2}$ | 1060 | 865 | 445 | 0.54 | 0.51 | 6.4 |
| 2 | 1S..... | 3-oval-core | Cinders | $1:1:4\frac{1}{2}$ | 1060 | 865 | 350 | 0.43 | 0.40 | 5.0 |
| 3 | 2F..... | 3-oval-core | Haydite | $1:1:4\frac{1}{2}$ | 1060 | 820 | 455 | 0.54 | 0.56 | 6.5 |
| 4 | 2S..... | 3-oval-core | Haydite | $1:1:4\frac{1}{2}$ | 1060 | 820 | 370 | 0.43 | 0.45 | 5.3 |
| 5 | 3F..... | 3-oval-core | Sand-Gravel | $1:1:4\frac{1}{2}$ | 1060 | 1070 | 610 | 0.54 | 0.57 | 8.7 |
| 6 | 3S..... | 3-oval-core | Sand-Gravel | $1:1:4\frac{1}{2}$ | 1060 | 1070 | 450 | 0.43 | 0.42 | 6.4 |
| 7 | 4F..... | 3-oval-core | Limestone | $1:1:4\frac{1}{2}$ | 1060 | 780 | 390 | 0.54 | 0.50 | 5.6 |
| 8 | 5F..... | 3-oval-core | Slag | $1:1:4\frac{1}{2}$ | 1060 | 1290 | 540 | 0.54 | 0.42 | 7.7 |
| 9 | 6F..... | 3-oval-core | Haydite | $1:1:4\frac{1}{2}$ | 1060 | 1280 | 440 | 0.54 | 0.34 | 6.3 |
| 10 | 7F..... | 3-oval-core | Cinders | $1:1:4\frac{1}{2}$ | 1060 | 970 | 515 | 0.54 | 0.53 | 7.4 |
| 11 | 8F..... | 3-oval-core | Sand-Gravel | $1:1:4\frac{1}{2}$ | 1060 | 1230 | 695 | 0.54 | 0.57 | 9.9 |
| 12 | 9F..... | 3-oval-core | Cinders | $1:\frac{1}{4}:3$ | 1920 | 1150 | 610 | 0.54 | 0.53 | 7.6 |
| 13 | 10F.... | 3-oval-core | Sand-Gravel | $1:\frac{1}{4}:3$ | 1920 | 550 | 355 | 0.54 | 0.65 | 4.4 |
| 14 | 11F.... | 3-oval-core | Sand-Gravel | $1:\frac{1}{4}:3$ | 1920 | 1570 | 780 | 0.54 | 0.50 | 9.6 |
| 15 | 12F.... | 5 by 8 by 12 in.<br>2-square-core | Sand-Gravel | $1:1:4\frac{1}{2}$ | 1060 | 1360 | 585 | 0.37 | 0.43 | 8.3 |
| 16 | 13F.... | $3\frac{1}{2}$ by 3 by 12 in.<br>2-square-core | Sand-Gravel | $1:1:4\frac{1}{2}$ | 1060 | 1550 | 630 | 0.56 | 0.41 | 9.0 |
| 17 | 14F.... | 8 by 12 by 16 in.<br>3-oval-core | Sand-Gravel | $1:1:4\frac{1}{2}$ | 1060 | 600 | 370 | 0.52 | 0.62 | 5.3 |
| 18 | 15F.... | 3-oval-core | Haydite | $1:1:4\frac{1}{2}$ | 1060 | 670 | 365 | 0.52 | 0.54 | 5.2 |

[a] See Table IV, of the paper by F. E. Richart, P. M. Woodworth and R. B. B. Moorman on "Tests of the Stability of Concrete Masonry Walls," see p. 671.

[b] Based on maximum allowable working load—70 lb. per sq. in. for cement-lime mortar and 80 lb. per sq. in. for cement mortar as specified in "Recommended Minimum Requirements for Masonry Wall Construction" (1924).

The concrete masonry walls tested were 6 ft. wide, 9 ft. 6 in. high, and 8 or 12 in. thick, approximately the same size as those considered by Mr. Parsons in the development of his formula. The data on the strength of the mortar and of the individual units were obtained in a similar manner in both investigations. The accompanying Table I summarizes the tests on eighteen groups of walls, eleven of which are the average of three individual walls (References 1 to 11) and seven, the average of two walls (References 12 to 18), a total of 47 walls.

The range in wall strengths was from 350 to 780 lb. per sq. in. with a corresponding range in factor of safety from 4.4 to 9.9. In all cases the walls have a strength substantially in excess of the 280 and 320 lb. per sq. in. mentioned previously.

The actual strengths of the walls as determined in the tests are plotted against strengths calculated by the formula $M = b\sqrt{mu}$ in the accompanying Fig. 1. It will be seen that there is a marked similarity between the actual strengths and the calculated strengths for concrete masonry and those in Fig. 3 of Mr. Parson's paper for clay tile which are also plotted in the accompanying Fig. 1 for purpose of comparison.

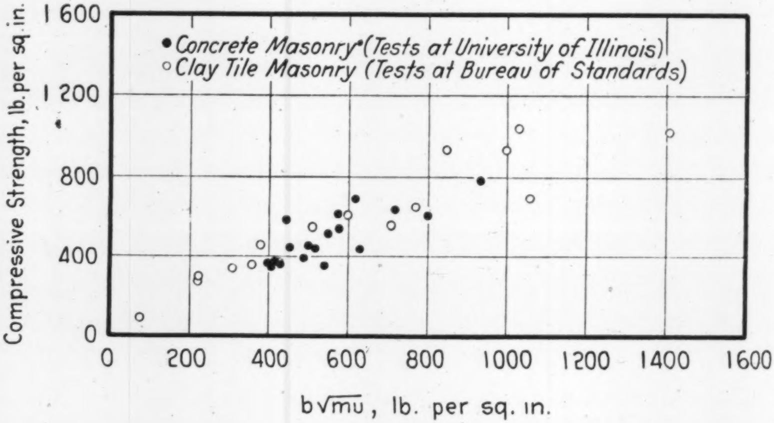

FIG. 1.—Relation Between Compressive Strength (Gross Area) of End-Construction Masonry and $b\sqrt{mu}$.

The formula developed by the University of Illinois' tests, $W = 0.53\ u$, for 8 by 8 by 16-in. units with full mortar bedding is substantially the same as $M = bu$, as the ratio of bedded area to gross area was about 0.54. In the case of the face shell bedding, the University of Illinois' tests show that the wall strengths were also closely proportional to the bedded area. The formula $M = bu$, then, corresponds to Mr. Parsons' formula $M = b\sqrt{mu}$ when the range in strengths between $m$ and $u$ is not great. Tests are under way to investigate further the effect on wall strength of a wide range in strength of mortar and strength of units. In these tests, concrete masonry units ranging in strength from 400 to 4000 lb. per sq. in. will be laid up with mortars ranging in strength from 200 to 3000 lb. per sq. in.

In Mr. Parsons' formula, wall strengths are directly proportional to the ratio of bedded to gross area. The University of Illinois' tests on concrete masonry walls show a similar relation. For the more commonly used types of concrete masonry units, the ratio of bedded to gross area with both face shell and full mortar bedding are as follows:

| Type of Unit | Minimum Face Shell Thickness, in. | Area, sq. in. |  | Approximate Percentage of Gross Area in Bearing |  |
|---|---|---|---|---|---|
|  |  | Gross | Net | Face-Shell Bedding | Full-Mortar Bedding |
| 8 by 8 by 16 in. 3-oval-core | 1½ | 126 | 75 | 43 | 50 to 60 |
| 8 by 8 by 16 in. 2-square-core | 1 to ⅞ | 126 | 88 | 47 | 60 |
| 5 by 8 by 12 in. 2-square-core | 1 | 89 | 38 | 26 | 37 |
| 3½ by 8 by 12 in. 2-square-core | 1¾ | 89 | 62 | 45 | 56 |

It will be seen from the tabulation that the area available for mortar bedding varies from 26 to 47 per cent for face shell bedding and from 37 to 60 per cent for full mortar bedding. The four types of concrete masonry units shown represent 95 per cent of the present production.

The use of the approximate formula $M = b\sqrt{mu}$ will be helpful to masonry unit manufacturers, both concrete and clay, in estimating the bedding area and strength of unit required to develop the necessary wall strengths. The direct relation developed at the University of Illinois between the strengths of full size walls, $W$, (6 by 9½ ft.) and wallettes, $S$, (32 by 48 in.) and expressed by the formula $W = 0.91\,S$ affords the specification writer a simple test specimen with which to check design limitations of concrete masonry determined by the use of the formula $M = b\sqrt{mu}$.

The writer believes that the equation developed by Mr. Parsons will serve to focus attention on the fact that there are other factors in addition to strength of unit which influence the strength of end-construction masonry, the most important of which is the ratio of bedded to gross area.

MR. BENJAMIN WILK.[1]—In considering Mr. Parsons' formula in the light of the results presented by Messrs. Richart, Woodworth and Moorman on thin wall shells of Haydite block, I am wondering how far the formula holds true on thin wall shells of concrete units. In Mr. Richart's results the bedded area of the two types of block differed by 20 per cent but the strengths differed by only 10 per cent. In some tests made by us in Detroit on units of bedded area of 60 per cent, we found the strengths equivalent to 80 per cent of the full bedded strength. It strikes me that perhaps the area bedded near the center of the unit is less effective than the area bedded away from the center of the unit. Inasmuch as considerable work is being done with concrete units with thin wall shells, it would probably be desirable to have further tests made to see how the results compare with Mr. Parsons' formula.

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[1] Manager, Standard Building Products Co., Detroit, Mich.

MR. D. E. PARSONS[1].—The paper by Mr. Menzel and the one by Messrs. Richart, Woodworth and Moorman are outstanding contributions to this subject. They supply the kind of information that is needed for the development of satisfactory regulations governing the use of concrete units.

These two papers both show a linear relationship between the strengths of concrete masonry walls and of the concrete units, whereas the data analyzed in my paper indicate strongly that the relationship between the strength of the walls and the strength of the units is not a linear one.

Several of the speakers have suggested possible reasons for the difference. Mr. Small discusses the influence of the stresses in the mortar and suggests that specifications may be of value indirectly as well as directly.

In the paper by Richart, Woodworth and Moorman, mention is made of certain other so-called minor variables that are not taken into account ordinarily. As a rather minor but interesting sidelight of the data presented in this paper, it is shown that the ratio between the strength of the masonry and the strength of the units is in general somewhat greater the weaker the mortar cylinders. This, of course, does not mean that the weaker the mortar the stronger the wall, but it simply emphasizes some of the variables that cannot be evaluated because of the lack of precision in the estimates of the properties of the mortars from measures of the strengths of the mortar cylinders.

I greatly appreciate Mr. Gonnerman's discussion. He not only has presented an analysis of new data but has emphasized the theme that factors other than the strength of the unit have an influence on wall strength. It is particularly fortunate that the new test results analyzed by Mr. Gonnerman are being presented before the formula given in the paper had reached an age that might tend to give it authority. The lack of precision in representing data, both the old and the new, reflects the incompleteness of Eq. 1. While the equation was a useful tool in analyzing the test data of the paper, it represents only a small start toward an evaluation of the factors controlling the strength of end-construction masonry of hollow units. It is hoped that it may be replaced soon by one that is more complete.

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[1] Chief, Masonry Construction Section, U. S. Bureau of Standards, Washington, D. C.

MR. H. S. BRIGHTLY.[1]—I should like to call attention to one of the points brought out in Mr. Menzel's paper. He showed that the substitution of a considerable percentage of lime for cement in the mixtures used did not make any appreciable difference in the strength of the mortar. I want to say that that fact has been confirmed to a large extent by a series of tests on stone-setting mortars conducted at the U. S. Bureau of Standards by a research associate of the former Indiana Limestone Quarrymen's Assn., the results of which investigation unfortunately have not as yet been published.

I should like to ask Mr. Dutton, who conducted these tests, to explain the results obtained. I may state briefly, however, it was found that substitutions of from 30 to 35 per cent of lime for the cement content of these mortars gave really the best mortars for stone setting purposes—mortars that had higher adhesive strength and consequent higher bonding values and excellent working and spreading properties, without appreciable loss in compressive strength, and that substitutions of lime even as high as 40 per cent did not cause a reduction in the strength great enough to be considered a real detriment to the mortar for stone-setting operations.

MR. H. H. DUTTON.—The tests referred to by Mr. Brightly were made to determine the adhesive strengths of 1:3 mortars when used for setting stone and in that work we also included the determination of the tensile and compressive strengths for the same mixes. Hydrated lime was used to replace certain percentages of the cement content in the various batches and it was found that the mortars having from 30 to 40 per cent of the cement replaced by lime gave the best combinations of adhesive, tensile and compressive strengths. Such mixes, while naturally lower in strength than straight 1:3 portland-cement mixes, were considered entirely satisfactory for stone setting purposes and presumably should also be generally useful for laying up other masonry units.

MR. F. E. RICHART[2] (*authors' closure by letter*).—Since the completion of the wall tests described in our paper, we have tested another group consisting of eight walls, in cooperation with the Western Brick Co. These walls were of the same size as the large walls described in the paper, and were made and tested by the same staff. The walls were made with two styles of Haydite building units, having thinner face shells than the 37-per-cent core units used previously. One unit was of the 3-oval-core type, having 45 per cent core space, and the other was of the 3-rectangular-core type, having about 55 per cent of core space. With each type of unit, two plain walls were built and two composite walls, using a shale face brick. A header course forming each seventh course of the brick walls was backed up with Haydite brick. The walls were made with units furnished when about 28 days old, the oval core units being made by the Michigan Silo Co., and the rectangular core units by the W. G. Traver Co. The walls were tested 32 days after being laid up. The tests were made in the Southwark-Emery Machine, following in all respects the procedure of the previous tests. A summary of the principal results is given in the accompanying Table II.

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[1] Secretary-Director, Building Stone Association of Indiana, Inc., Bloomington, Ind.

[2] Research Professor of Engineering Materials, University of Illinois.

TABLE II.—TESTS ON HAYDITE WALLS MADE WITH THIN-WALLED UNITS.

| Wall | Type of Unit | Aggregate | Mortar Mix by Volume, Dry Rodded | Maximum Load |  | Compressive Strength, lb. per sq. in |  | Modulus of Rupture, lb. per. sq. in. | Ratio of Strength of Wall to Strength of Unit |
|---|---|---|---|---|---|---|---|---|---|
|  |  |  |  | lb. | lb. per. sq. in. | Units | Mortar |  |  |
| A 1........ | 8 by 8 by 16 in. 3-oval-core 45 per cent | Haydite.... | $1:1:4\frac{1}{2}$ | 234 000 | 400 | ... | 1220 | .. | .... |
| 2....... |  |  |  | 244 000 | 420 | ... | 1220 | 40 | .... |
| Average..... |  |  |  | 239 000 | 410 | 805 | 1220 | .. | 0.51 |
| B 1........ | 8 by 8 by 16 in. 3-rectangular-core 55 per cent | Haydite.... | $1:1:4\frac{1}{2}$ | 202 000 | 355 | ... | 860 | .. | .... |
| 2........ |  |  |  | 213 000 | 375 | ... | 860 | 40 | .... |
| Average..... |  |  |  | 207 500 | 365 | 720 | 860 | .. | 0.51 |
| C 1........ | 8 by 8 by 16 in 3-oval-core 45 per cent | Haydite and brick..... | $1:1\frac{1}{4}:4\frac{1}{2}$ | 674 000 | 755 | ... | 1260 | .. | .... |
| 2........ |  |  |  | 580 000 | 650 | ... | 1125 | 39 | .... |
| Average..... |  |  |  | 627 000 | 700 | 805 | 1190 | .. | 0.87 |
| D 1........ | 8 by 8 by 16 in. 3-rectangular-core 55 per cent | Haydite and brick..... | $1:1\frac{1}{4}:4\frac{1}{2}$ | 390 000 | 435 | ... | 1345 | 31 | .... |
| 2........ |  |  |  | 775 000 | 865 | ... | 1600 | .. | .... |
| Average..... |  |  |  | 582 500 | 650 | 720 | 1470 | .. | 0.90 |

It is of interest to note that the ratio of the strength of wall to strength of individual unit was 0.51 for the plain walls using both types of units. This value is very close to the average value of 0.53 found from the previous tests. The strengths of the composite walls also compare very favorably with the strengths of composite walls of lots 16 and 17. It might be concluded from this series of tests that the thin-shell units will give strength results comparing very well with those of the heavier units.

Mr. Gonnerman has shown that the results of the main series of tests agree quite well with Mr. Parsons' equation for wall strength. Admitting that this agreement exists, it is difficult to see why the ratio, *b*, of bedded to gross areas enters the strength relation, especially for fully bedded walls. For walls with face shells bedded only, one might expect the ratio of bedded area to net area to enter, but with full bedding the conditions are similar for walls and units, since the gross area is used in computing the strength of both. The difference in strength between units and walls may be explained by the difference in end restraint of units and by the variation in the thickness and alignment of the joints. The mortar joints in a wall vary in thickness; in the block molded on a metal pallet there is a groove in the bottom face which produces a considerable difference in the thickness of mortar across the shell. Strain gage measurements on walls showed a much greater deformation across the mortar joint than that within a unit. Hence, with a large deformation in the mortar, the load tends to concentrate on the thinner part of the joint away from the groove and near the faces of the shells. In many cases it was observed that a wall failure began by the splitting off of a portion of the outer shell at the bottom of a unit. In the test of a single unit with a thin cap of plaster of Paris, there is much less chance for this sort of failure, since the frictional restraint offered by the platens of the testing machine adds greatly to the resistance of the material to initial breakdown. This test is also free from the variations in mortar joint thickness which occur in a wall and produce local stress concentrations.

As noted by Mr. Wilk, the walls made with thin-shelled units show as great strength relative to that of the unit as did those made with thick-shelled units. With regard to the units with 55 per cent of rectangular cores, two compensating factors evidently entered. The bedding of end webs was evidently imperfect since two end webs rested above an interior web of the course below; on the other hand, these units were free from the groove in the bottom bearing face. These thin-shelled units failed to agree with Parsons' equation; one might consider what would happen in the opposite extreme. If a wall were laid up of solid blocks (for which the ratio of bearing to gross area, $b$, is 1.0), using a mortar stronger than the units, one would still expect the wall strength to be considerably less than that of the unit; probably 25 per cent less for walls of the size and slenderness used in the foregoing tests. This is merely in accord with test results from plain concrete columns with no mortar joints.

The foregoing paragraph indicates that there may be limitations on the use of Parsons' equation; at the same time the authors wish to express their appreciation of Mr. Parsons' thorough study of the question of wall strength and of his paper, which has been especially stimulating and instructive. They also wish to thank Mr. Gonnerman for adding very greatly to the analysis of the information given in the paper.

# THE EFFECT OF TIME LOADING UPON THE BOND STRESS BETWEEN CONCRETE AND STEEL

By R. L. Brown[1] and E. C. Clark[2]

## Synopsis

These tests were made to study the effect of time loading upon the bond stress between concrete and steel. Fifty-two beams and twelve pull-out specimens were tested. Four beams were tested as a preliminary set. The time test consisted of twenty-eight beams. The position-of-molding series consisted of twelve beams. Eight beams were made to tie in the results of the time test with those obtained in the usual method of testing beams. Strains were measured along the reinforcement bar. Slip readings were taken at three points on each end of the beam. Deflections were measured on each side of a beam. The main series of beams were under load from 18 to 20 weeks.

The time test showed a gradual increase in deflection, a gradual increase in end slip, and a gradual increase of stress in the reinforcement bar. Beams poured in an inverted position gave values about 20 per cent lower than beams poured in the usual manner. The use of spring loading in the time test gave results comparable with the usual machine loading. The preparation of a beam to take strain readings on the reinforcing steel reduced the strength about 20 per cent.

The sliding resistance was the important part of the bond resistance in these tests. The adhesive resistance was in most cases destroyed almost to the end of the beam quite early in the test. Because of this the deformed bars were much superior, carrying more load with less end slip.

## Introduction

The tests here reported were made in 1929 to study the effect of time loading upon the bond stress between concrete and steel. Beams were chosen as test specimens because they produce a distribution of bond stress that is representative of reinforced-concrete flexural members and because of their adaptability to time tests. The tests were suggested by a few tests of beams under sustained loading made in this laboratory in 1915 by Mr. R. E. Turley.[3]

Bond resistance, as analyzed by Abrams in *Bulletin 71* of the Engineering Experiment Station, University of Illinois, may be divided into two parts; namely, adhesive resistance and sliding resistance. Adhesive resistance is used to designate the bond resistance developed before movement of the bar with respect to the adjacent concrete begins. Sliding resistance is the term applied to the resistance developed as a result of the movement of the bar. Adhesive resistance may be considered as due to tangential adhesion between the concrete and steel and to static friction. A bar in the shape of a perfect cylinder must have a very low sliding resistance, while a deformed bar will have a high value.

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[1] Research Associate, Engineering Experiment Station, University of Illinois, Urbana, Ill.

[2] Instructor, Department of Mechanics, Ohio State University, Columbus, Ohio.

[3] R. E. Turley, "Bond Stress in Reinforced Concrete Beams," Master's thesis, University of Illinois, 1915.

These tests were made in the Concrete Research Laboratory of the Department of Theoretical and Applied Mechanics, as a project of the Engineering Experiment Station, University of Illinois.

The tests were made in four groups as follows:

*Series 0.*—Four beams, tested in the testing machine.

*Series 1.*—Twenty-eight beams, tested under time loading.

*Series 2.*—Twelve beams, tested to study the effect of position of molding.

*Series 3.*—Eight beams and twelve pull-out specimens, to study the effect of loading through springs.

The tests of series 0 were made as a preliminary to series 1 to determine what loads should be used in that series, and to develop testing technique.

In series 1, tests of beams under sustained loading comprised the main portion of the tests. Of the 28 beams in this series, 16 were made with plain round reinforcement bars and 12 with deformed bars. The load was maintained on these beams by car springs used in a special loading rig.

In series 2, a study was made to determine the effect of the position of molding upon the value of bond resistance developed. Six beams were made with the reinforcing steel near the top of the form as the beam was poured, and six beams were made in the usual position with the steel near the bottom of the beam. All of these beams were tested in a testing machine, with spring loading.

The tests of series 3 were made to show the relation between beams as tested under spring loading in series 1 and those tested in the usual way in a screw power testing machine. No strain readings were taken on the reinforcing steel in this series but the comparison was made on the basis of ultimate loads carried. There were eight beams, all with plain round reinforcement bars. Four beams were tested with springs between the head of the machine and the beam, while the other four were tested in the usual manner. This series also contained twelve pull-out specimens; six were made with plain round bars, and six with deformed bars. Half of the specimens of each kind were tested with a spring between the concrete and the base of the machine instead of the usual spherical bearing block.

## MATERIALS

*Steel.*—One-inch round bars were used in all of the specimens. The physical properties of the plain round and deformed bars were not as nearly alike as was desired, although the yield point stress was not exceeded in any of the bond tests. Properties of the steel are as follows:

| Kind of Bar | Yield Point, lb. per sq. in. | Tensile Strength, lb. per sq. in. | Elongation in 8 in., per cent | Reduction of Area, per cent |
|---|---|---|---|---|
| Plain | 39 200 | 60 900 | 29.8 | 58.6 |
| Deformed | 52 500 | 89 500 | 22.5 | 39.2 |

*Concrete.*—All concrete materials passed the standard tests. Universal cement and Attica sand and gravel were used in the concrete. The mix used was 1:2.7:3.75, by loose volume, with a water-cement ratio of 0.90. The concrete was mixed four minutes in a "Wonder" batch mixer. The average strength of 6 by 12-in. compression cylinders was 3700 lb. per sq. in. at the time the beams were loaded. All specimens were cured in the damp room for 23 to 25 days. The concrete was 28 to 30 days old when the specimens were loaded.

## TEST METHODS

Beams were used as test specimens in all series, and series 3 also contained twelve pull-out specimens.

The beams were all of the same nominal size, 6 by 12 in. by 6 ft. 6 in., having an effective depth of 10 in. and a span of 6 ft. The tensile reinforcement consisted of one 1-in. round bar. Diagonal reinforcement consisted of twelve ½-in. plain round loop stirrups spaced 4 in. apart. Each beam also contained the following short bars: three short ½-in. bars in each end forming a plane of reference from which end slip was measured and three ½-in. square bars transversely through the beam, one projecting 2 in. on each side at each end, and one near the center from which deflections of the beam were measured.

Except in series 3 where the strain gage readings were omitted, the following readings were taken on each beam:

Eleven strain readings on the steel with a 4-in. gage.

Four slip readings between the concrete and steel with a 4-in. gage, two at the load points and two at a section 9-in. from the end of the beam.

Two slip readings, one at each end of the beam.

Two deflection readings, one on each side of the beam.

The beams in series 0, 2 and 3 were tested one at a time in a testing machine. Readings were taken at loads of 5000 lb., 10,000 lb., and each 1000 lb. thereafter until failure of the beam. In beams with deformed bars an increment of 2000 lb. was used above the 10,000-lb. load.

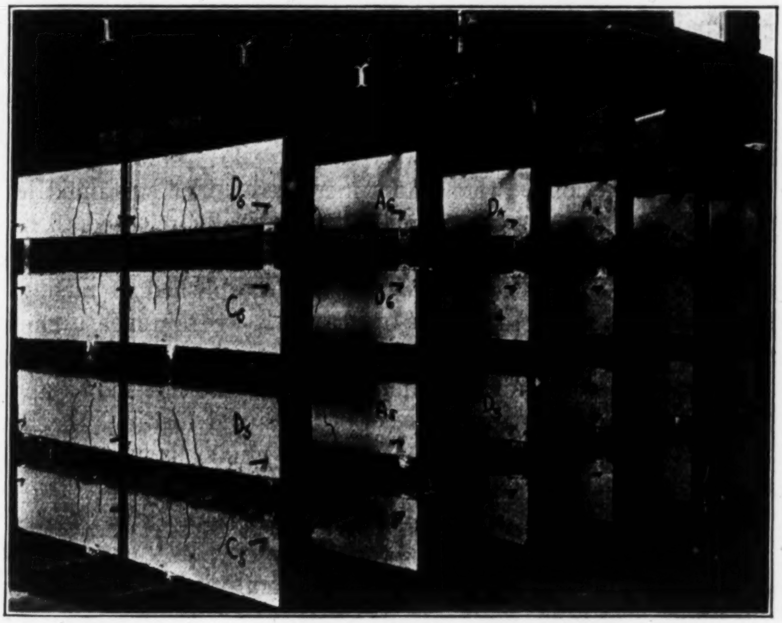

FIG. 1.—Beams of Series 1 Under Load.

In series 1, the time test, the beams were placed four in a stack, one above the other, with supports arranged to produce third-point loading, and the stack was then loaded by means of the special car-spring apparatus shown in Fig. 1. With this system of loading half of the beams were tested upside down, the effective dead load (including apparatus) on each beam being:

| | |
|---|---|
| Top beam | 375 lb. |
| Second beam | 667 lb. |
| Third beam | 1375 lb. |
| Bottom beam | 1667 lb. |

The bottom two beams have 1000 lb. more load than the corresponding ones at the top of the stack. This fact was used to obtain a 1000-lb. increment of load on the beams with a 2000-lb. increment on the spring apparatus.

From the results of the tests of series 0, spring loads were chosen to produce 15,000, 17,000 and 19,000 lb. on the top beam of each stack, thus providing loads on six pairs of identical beams of about 15,000, 16,000, 17,000, 18,000, 19,000 and 20,000 lb. These loads were applied to the beams made with deformed bars, but since three of the beams with plain bars failed while the 19,000-lb. spring load was being applied, this group was replaced by four more beams under a spring load of 13,000 lb.

Preparatory to use, the car springs were carefully calibrated in a testing machine, shortening of the spring being measured on gage lines on each side. From this calibration, the loads were applied to each stack of beams by tightening the tie-rods by hand until the desired shortening of the spring was reached.

As the deflection of the beams increased under sustained load, the spring load was correspondingly automatically released to some extent. After the first few weeks of loading, during which the release of load was considerable, the practice was established of raising all spring loads to their original value once a week. Strain measurements were taken immediately after these readjustments of load.

Since the beams made with deformed bars had not shown appreciable end slip after 10 to 12 weeks of sustained loading, the loads on two stacks were raised from 15,000 and 17,000 lb. to 23,000 and 21,000 lb., respectively.

The beams in this series were under load from 18 to 20 weeks. Since it was necessary to vacate the laboratory at the end of this period, the tests were discontinued. A general view of the group of beams under load is shown in Fig. 1.

The pull-out specimen consisted of a bar about 30 in. long embedded at one end in an 8 by 8-in. concrete cylinder. Two $\frac{1}{2}$-in. bars were placed in the upper end of the cylinders to carry a dial which was used to measure slip of the bar. An 0.0001-in. Ames dial was attached to these two bars with the plunger resting on a ball bearing set in the end of the main bar. An increment of load of 1000 lb. was used in testing the specimens. There were 6 pull-out specimens made with plain bars and 6 with deformed bars. In half of each lot the load was transmitted through a spherical bearing block and in the remainder it was applied through a car spring. A faster machine speed was used in conjunction with the spring bearing than with the spherical bearing block to make the total time of test about even for the two methods.

## TEST DATA

Figure 2 gives results of the sustained loading tests for a typical beam of series 1. The figure shows the variation in stress on each gage line for beam D6 as the load was continued for 18 weeks. The lower diagram also shows the variation in the tensile stress along the reinforcement bar. This beam was reinforced with a deformed round bar and was subjected to a spring load of 19,000 lb. The increase in average steel stress at midspan with continued load may

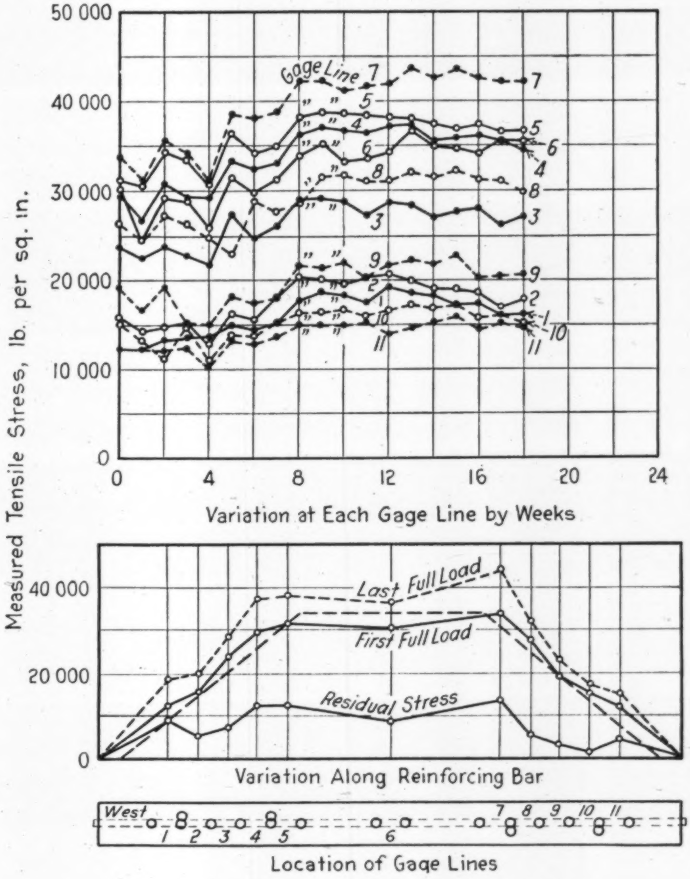

FIG. 2.—Stress Data of Beam D6 of Series 1.

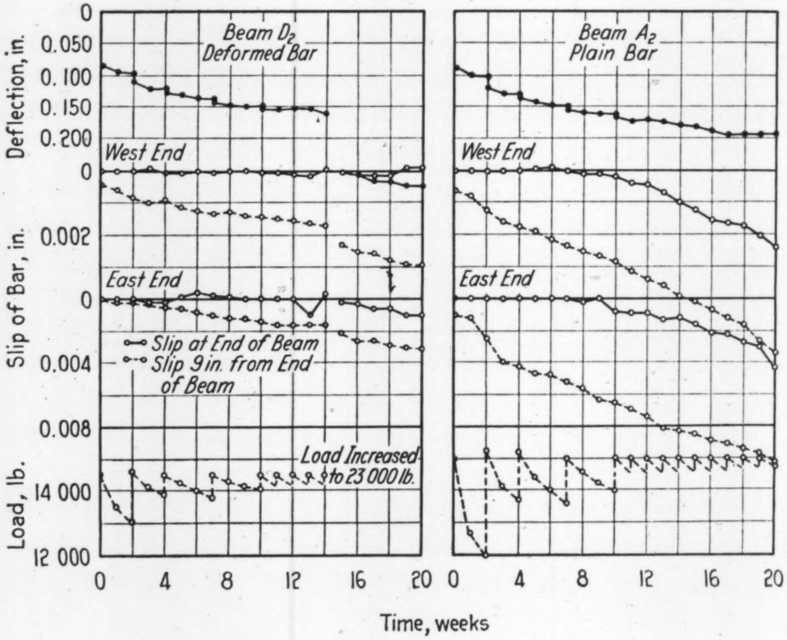

FIG. 3.—Slip Data of Beams D2 and A2 of Series 1.

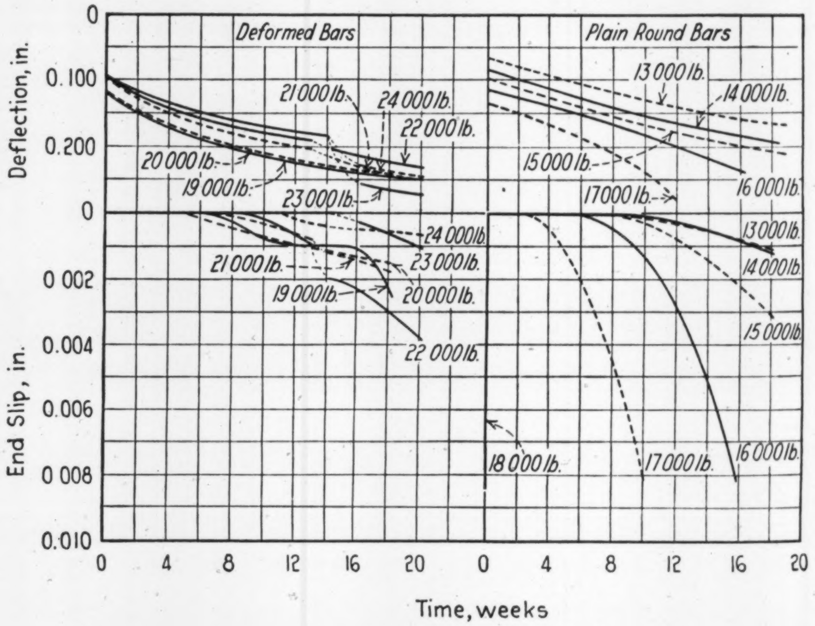

FIG. 4.—Average Results of Series 1.

be due partly to slip of bar; it seems more likely that it is caused by plastic yielding of the concrete on the tension side of the beam.

Figure 3 shows the variation in end slip of bar, the slip 9 in. from the end of the beam, the deflection and the load for beams A2 and D2 of series 1. The two beams, made with plain and deformed bars, respectively, were held under a load of 15,000 lb. for the major part of the time.

Figure 4 shows average curves which embody the principal results of the tests of series 1. End slips and deflections for both types of beams as observed with various sustained loads are shown. The increase in deflection up to 12 weeks' loading is quite similar for beams with plain and deformed bars. This increase is considerable even in beams in which little end slip has occurred; it may be due in part to progressive slip of bars in the outer thirds of the beam resulting in a gradual increase in the stress and the resulting deformation of the bar, and in part to plastic flow of the concrete in flexure.

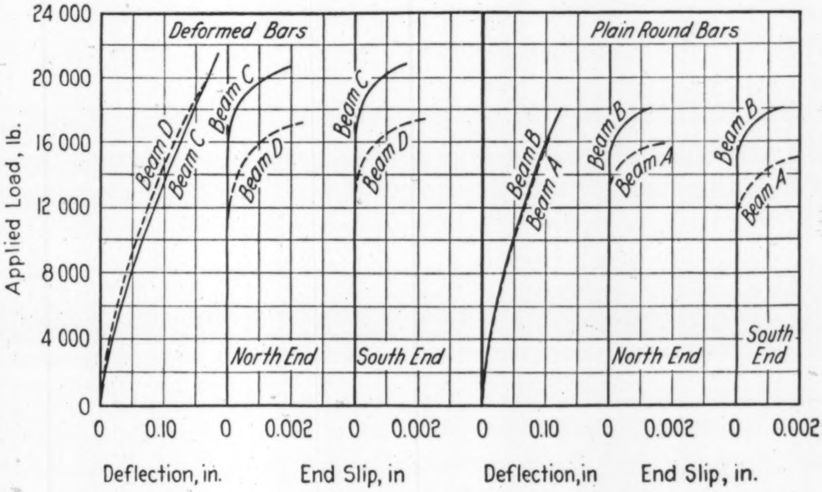

FIG. 5.—Average Results of Series 2 Beams A and D Poured in Inverted Position.

The end slips for beams with deformed bar reinforcement are seen to be generally much less than for those with plain bars, under comparable loads.

A comparison of the results of series 2 is given in Fig. 5, in which average values of the deflection and end slip are given. The general results of series 2, as well as those of series 0 and series 3, are given in Tables I and II.

## DISCUSSION OF RESULTS

In the tests of series 1, three closely related effects were observed. In general, with sustained loading there was a continued increase in deflection of the beam, in slip of bar at the end of the beam, and in the measured tensile stresses in the reinforcement bar. The deformed bars showed less end slip, even with higher loads, than the plain bars. It is quite evident that there was enough slip in the bar near the load point for the deformed bar to develop very high bond resistance. When the load was removed from the beams the amount of strain remaining in the reinforcement bars was in most cases about equal to the gain in tensile strain during the period of sustained loading.

TABLE I.—DATE OF BEAMS TESTED IN SERIES 0, 2 and 3

| Series | Beam Number | Main Reinforcing | Type of Loading | Maximum Load, lb. | Bond Stress, lb. per sq. in. |
|---|---|---|---|---|---|
| 0 | A-0 | Plain round | Usual | 20 000 | 370 |
| | B-0 | Plain round | Usual | 24 000 | 440 |
| | C-0 | Deformed | Usual | 30 000 | 550 |
| | D-0 | Deformed | Usual | 25 000 | 460 |
| 2 | A-21[a] | Plain round | Spring | 14 000 | 260 |
| | A-22[a] | Plain round | Spring | 15 000 | 280 |
| | A-23[a] | Plain round | Spring | 16 000 | 290 |
| | B-21 | Plain round | Spring | 18 000 | 330 |
| | B-22 | Plain round | Spring | 18 000 | 330 |
| | B-23 | Plain round | Spring | 12 000 | 220 |
| | C-21 | Deformed | Spring | 26 000 | 480 |
| | C-22 | Deformed | Spring | 26 000 | 480 |
| | C-23 | Deformed | Spring | 26 000 | 480 |
| | D-21[a] | Deformed | Spring | 20 000 | 370 |
| | D-22[a] | Deformed | Spring | 24 000 | 440 |
| | D-23[a] | Deformed | Spring | 24 000 | 440 |
| 3 | A-31 | Plain round | Usual | 21 000 | 390 |
| | B-32 | Plain round | Usual | 21 000 | 390 |
| | B-33 | Plain round | Usual | 21 000 | 390 |
| | A-34 | Plain round | Usual | 22 000 | 410 |
| | B-31 | Plain round | Spring | 22 000 | 440 |
| | A-32 | Plain round | Spring | 23 000 | 420 |
| | A-33 | Plain round | Spring | 21 000 | 390 |
| | B-34 | Plain round | Spring | 20 000 | 370 |

[a] These beams were poured in inverted position.

TABLE II.—DATA OF PULL-OUT SPECIMENS OF SERIES 3.

| Series | Number | Type of Bar | Type of Loading | Load, lb. | Bond Stress, lb. per sq. in. | Slip, in. |
|---|---|---|---|---|---|---|
| 3 | PS-1 | Plain round | Usual | 13 000 | 520 | 0.001 |
| | PS-2 | Plain round | Usual | 13 500 | 540 | 0.001 |
| | PS-3 | Plain round | Usual | 13 000 | 520 | 0.002 |
| | PS-4 | Plain round | Spring | 13 000 | 520 | 0.001 |
| | PS-5 | Plain round | Spring | 16 000 | 640 | 0.002 |
| | PS-6 | Plain round | Spring | 15 000 | 600 | 0.0014 |
| | PC-1 | Deformed | Usual | 15 500 | 620 | 0.0015 |
| | PC-2 | Deformed | Usual | 15 000 | 600 | 0.0015 |
| | PC-3 | Deformed | Usual | 14 000 | 560 | 0.002 |
| | PC-4 | Deformed | Spring | 15 500 | 620 | 0.0018 |
| | PC-5 | Deformed | Spring | 16 000 | 640 | 0.002 |
| | PC-6 | Deformed | Spring | 14 000 | 560 | 0.0019 |

This indicates that the gradual increase in measured tensile stress may have been due to a considerable extent to plastic flow in the uncracked sections of concrete in tension as well as to general slip of bar. Both of these effects would tend to bring the average stress on the 4-in. gage length up to the maximum value.

Since slip of bar is known to progress from load point toward the support in beams of the type used, the occurrence of appreciable end slip is an indication of general slipping of bar in the outer thirds of the beam. Increasing end slips may be taken as evidence that the bond developed is due to sliding resistance. It is obvious that with any large slip the deformed bars should develop a large resistance of this sort, as indicated by the larger loads carried in beams with deformed bars. The tests raise some question as to whether the criterion of the critical amount of slip, as found by the ordinary rapid test to failure, is valid. In beams well reinforced against diagonal tension failure, as these were, the deformed bars show a superiority to plain bars not generally indicated in the short-time test.

The results of series 2 were much as would be expected, judging from previous tests.[1] With the reinforcement bar near the upper surface of the beam during casting, if it is held rigidly and not allowed to settle with the concrete, there is a tendency for water and air to collect against the lower side of the bar, and in some cases the concrete actually settles away from the bar. The result is a decrease in the bond stress as compared with the bar cast in the lower part of the beam. The decrease in these tests was between 15 and 20 per cent.

In series 3 the bond strength of the beams loaded through spring supports agreed very well with that for beams loaded in the standard way. The beams of this series furnish information on the effect of cutting holes in the tension face of the beam to expose the reinforcement bar. The results of beams of this series in which the steel was not exposed, and those of series 2, in which it was exposed, show that the preparation of beams to take strain readings on the reinforcing steel reduces the bond stress about 20 per cent.

The results of the pull-out tests show little difference between the two methods of loading, using either a spring or a spherical bearing block. The results are consistent for both types of reinforcement bar. The bond strengths developed in the pull-out tests are in good accord with previous tests.

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[1] L. N. Edwards and H. L. Greenleaf, "Experimental Tests of Concrete-Steel Bond," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 584 (1928); also University of Illinois Engineering Experiment Station *Bulletin 71*, p. 175.

## CONCLUSION

The tests demonstrated the feasibility of testing reinforced-concrete beams under sustained loading and of testing several beams simultaneously with one load-holding rig. Slips of reinforcement bars at the end of the beam and at intermediate points were satisfactorily measured by the use of portable instruments.

The tests showed that after the adhesive resistance between concrete and steel had been overcome, as indicated by appreciable end slip of bar, a further margin of strength was provided by sliding resistance. This sliding resistance, however, permitted a gradually increasing amount of end slip under a steadily applied load. For the beams tested, which were well reinforced against diagonal tension failure, the sliding resistance of deformed bars, having transverse deformations offering a considerable amount of bearing surface, was much superior to that of plain bars.

The tests may be considered as exploratory work, but it is quite evident that this sort of test approaches more nearly the conditions of structural members in service. It is believed that more knowledge regarding bond stress is needed at the present time and tests under some form of sustained loading give promise of providing information of a fundamental character.

# DISCUSSION

THE CHAIRMAN (*Mr. Cloyd M. Chapman*[1]).—This very interesting paper touches on one of the things that has been left open in the Joint Committee report for ten years. The 1921 report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete defined a deformed bar as one which would develop a bond stress 25 per cent greater than that developed by a plain bar, but did not specify any means to determine the bond resistance nor the age at which the comparison should be made. At the same time the report permitted 20 per cent higher bond stress using a deformed bar than when using a plain bar. We have had need for a test of this kind for a long time. It has always seemed that in a long-time test a deformed bar would show up to much greater advantage than in a short-time test. The work described in the paper indicates that this is true. It is important, however, that the test be carried to failure under a long-time load, in order to get the true picture of the value of the deformed bar compared with the plain bar. It is generally, in fact almost always, in a beam or some flexural member that we have occasion to use a bond stress, and for that reason the standard test ought to be on a specimen somewhat like that described in the paper rather than on a "pull-out" test in which the results of the plain bars and deformed bars show very little difference. I hope that these tests will be carried further to failure, in order to bring out the ultimate relations between the bond resistance of plain bars and of deformed bars.

MR. F. E. RICHART.[2]—The paper describes a method of testing under long-continued loading now being used to a considerable extent in the study of such properties as bond resistance and plastic flow of concrete. Such a test approaches the conditions of actual service in a structure and obviously gives information to be obtained in no other way. As a matter of record, the tests by Messrs. Brown and Clark followed a line of study which was started at the University of Illinois by W. A. Slater some fifteen years ago. I believe he was the first to discover that in a beam reinforced with a straight bar, a load sufficient to produce an appreciable end slip of the bar would, if continuously applied, eventually produce failure.

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[1] Consulting Engineer, New York City.

[2] Research Professor of Engineering Materials, University of Illinois, Urbana, Ill.

The tests by Messrs. Brown and Clark were planned to provide a considerable range in intensity of bond stress, with the idea of determining how high a stress could be sustained indefinitely. While the tests were discontinued prematurely due to moving the laboratory, and few were carried to failure, the relative values of end slips, deflections and strains furnish useful information. I hope this paper may lead to further tests of this kind.

# COMPARATIVE TESTS FOR DETERMINING RESISTANCE OF FIRE-CLAY BRICK TO THERMAL SPALLING[1]

By R. A. Heindl[2]

## Synopsis

Specimens of twenty-three brands of fire-clay brick representing a fairly wide geographical distribution were furnished for these tests by twelve manufacturers of refractory products. The methods of manufacture included the semi-dry press, the stiff-mud extruded and the hand-made.

The brick were tested for resistance to thermal spalling in accordance with the standard method of testing promulgated by the Federal Specifications Board, that prescribed by this Society, and according to four methods which were modifications of these two standards.

The results show that, when the bricks are tested by the method given in the Federal specifications, those manufactured by the semi-dry press process are decidedly superior in resisting failure while those manufactured by the stiff-mud process show the least resistance. When using the A.S.T.M. method, the bricks manufactured by the hand-made process show the greatest resistance to failure with no significant difference between those manufactured by the semi-dry press and stiff-mud processes. The percentage mean deviation from the average in the number of cycles required to cause spalling is practically the same in the present tests. The percentage weight loss is decidedly greater in the A.S.T.M. method of test than in any of the other methods. However, the requirements of the method of testing account for the greater loss.

## Introduction

Many reports[3] have been published which discuss the theory of spalling action and the probable value of the water-dip quenching test for determining the resistance of fire-clay brick to thermal spalling in service.

At present two methods for testing the resistance of fire-clay bricks to thermal shock are generally recognized in this country. One was promulgated by the Society's Committee C-8 on Refractories, and the other by the Federal Specifications Board of the Federal

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[1] Publication approved by the Director of the Bureau of Standards of the U. S. Department of Commerce.

[2] Chief, Refractories Section, U. S. Bureau of Standards, Washington, D. C.

[3] U. S. Bureau of Standards *Technologic Paper No. 279*.

Am. Refractories Inst. *Technologic Paper No. 1*, and *Technical Bulletins Nos. 5, 24 and 26.*,

*Transactions*, Am. Soc. Mechanical Engrs., January–April, 1928.

*Journal*, Am. Ceramic Soc., Vol. 8, No. 1, p. 29 (1925); Vol. 8, No. 6, p. 361 (1925); and Vol. 9, No. 10, p. 654 (1926).

Government. In some cases consumers request that the bricks under purchase must comply with the requirements of the one test and in other cases with those of the other. Such a situation is not altogether desirable, because of the cost devolving on the manufacturer if he must have both tests made for purposes of plant control at regular intervals on all of his brands of brick. There is also the possibility of confusion on the part of the consumer in the preparation of his specifications. As a result it was decided to determine the relative merits of the present standard tests and, in so far as the Government is concerned, to determine whether or not it would be desirable to suggest changing the present Federal test for thermal spalling. The study also included tests using four modifications of the two standard tests and the data collected were correlated with the results obtained during the past several years in the Bureau's general investigation of fire-clay refractories.

TABLE I.—SOURCE OF BRICK, PROCESS OF MANUFACTURE AND NUMBER OF HEATING-AND-QUENCHING CYCLES NECESSARY TO CAUSE FAILURE OF SPECIMENS OF 23 BRANDS OF FIRE-CLAY BRICK.

Five bricks used except as indicated.

| Locality | Key Letter of Brand | Method of Manufacture | Cycles to Cause Failure | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Quenched from 850° C. | | | Quenched from 1350° C. | | |
| | | | Method 1[a] | Method 2[a] | Method 3[a] | Method 4[a] | Method 5[a] | Method 6[a] |
| Missouri..... | A....... | Semi-dry press.. | >88 | >64 | >100 | 88 | 19 | 40 |
| | B....... | Semi-dry press.. | 9 | 23 (22)[b] | 12 | 26 | 9 | 2 |
| | C....... | Semi-dry press.. | 29 | 36 | 30 | 42 | 8 | 7 |
| | D....... | Semi-dry press.. | 38 | 45 | 43 | 28 | 14 | .. |
| | E....... | Semi-dry press.. | 16 | 19 (18)[b] | 11[c] | 14 | 3 | .. |
| Pennsylvania. | F....... | Hand-made..... | 8 | >51 | 18 | 28 | 5 | .. |
| | G....... | Hand-made..... | 5 | 13 | 10 | 18 | 2 | .. |
| | H....... | Hand-made..... | 17 | 59 | 69 | 60 | 8 | 8 |
| | I........ | Semi-dry press.. | 15 | 25 (23)[b] | 19 | 28 | 4 | .. |
| | J....... | Hand-made..... | 10 | >58[d] | 90 | 50 | 7 | .. |
| | K....... | Stiff-mud....... | 3 | 16 (17)[b] | 20 | 37 | 6 | .. |
| | L....... | Semi-dry press.. | 10 | 53 | 25 | 23 | 11 | .. |
| Kentucky.... | M...... | Semi-dry press.. | 10 | 32 | 14 | 21 | 5 | .. |
| | N....... | Stiff-mud....... | 8 | 11 | 15 | 18 | 2 | 2 |
| | O....... | Stiff-mud ....... | 9 | 17 | 21 | 30 | 4 | .. |
| | P....... | Hand-made..... | 9 | 23 | 12 | 21[e] | 6 | .. |
| | Q....... | Hand-made..... | 12 | 42 | 27 | 30 | 7 | .. |
| | R....... | Hand-made..... | 10 | 48 | 23 | 39 | 3 | .. |
| California.... | S....... | Semi-dry press.. | 17 | 41 | 40 | 45 | 7 | .. |
| | T....... | Semi-dry press.. | 54 | 50 | 37[f] | 33 | 4 | 7 |
| Miscellaneous. | U....... | ............... | 34 | 52 | 67 | 49 | 11 | .. |
| | V....... | Semi-dry press.. | 21 | 11 | 15 | 13 | 2 | .. |
| | W...... | Stiff-mud repress | 30 | 43 | 20 | .. | 4 | 5 |
| Average loss in weight, per cent........... | | | 13 | 15 | 15 | 33 | 16 | .. |
| Mean deviation from the average number of cycles to cause failure, per cent.......... | | | 30 | 22 | 25 | 31 | 21 | .. |

[a] Salient points of difference in methods of testing summarized in a footnote to Table II.
[b] Figures in parentheses represent the average results of two 5-brick samples.
[c] Three bricks. [d] Four bricks. [e] Two bricks. [f] One brick.

## MATERIALS AND TEST METHODS

Specimens of twenty-three different brands of fire-clay brick, representing a fairly wide geographical distribution, were furnished by twelve manufacturers of refractory products. The methods of manufacture of these bricks included the semi-dry press, the stiff-mud extruded and the hand-made.

Sets of five bricks of each brand (except as noted in Table I) were tested in accordance with the following six methods:

*Method 1.*—As described in Federal Specifications for Fire-Clay Bricks.[1] The bricks receive a preliminary heating at 1400° C. for five hours. One end of each brick is then heated in the door of a suitable furnace maintained at 850° C. and at hourly intervals the hot end is immersed to a depth of 4 in. in a tank of running cold water. After removing and allowing the specimen to steam in the air for five minutes, the cycle is repeated until the entire end of the brick has completely spalled or until the structure of the brick has become so weakened that the end can be easily removed with the fingers.

*Method 2.*—The particulars of this test are identical with those described under method 1 except that the bricks are not given the preliminary heat treatment at 1400° C.

*Method 3.*—The bricks are placed in the door of a laboratory furnace which is gradually heated until a temperature of 1400° C. is obtained on the inside face of the panel of test bricks. After holding at this temperature for five hours, the bricks are allowed to cool slowly to atmospheric temperature. They are then quenched as described under method 1 except that the heated end is immersed in water to a depth of 2 instead of 4 in. Pieces are removed if the structure of the brick has become so weakened that it would not support the weight of the brick when held either vertically or horizontally by the weakened portion.

*Method 4.*—As described in the Society's Tentative Method of Test for Resistance of Fire-Clay Brick to Spalling Action.[2] The bricks in this case are not given a preliminary heat treatment, but one end is quenched from 1350° C. in 2 in. of running water. The heating and quenching cycle is continued until the bricks have lost 20 per cent in weight. The weakened portions are not removed with the fingers.

*Method 5.*—Similar to method 4 except that the weakened portions are removed with the fingers and the test discontinued as soon as the entire end has completely spalled.

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[1] Federal Specification No. HH-B-671.

[2] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 837 (1927); also 1930 Book of A.S.T.M. Tentative Standards, p. 202.

*Method 6.*—The bricks receive a preliminary heat treatment at 1400° C. for five hours and are then tested according to method 5. Only seven brands were tested in this manner.

For the convenience of the reader the salient points of difference between the six tests are summarized in a footnote to Table II.

In the preliminary heating at 1400° C. of either the whole brick or only one end, the entire sample was not fired at the same time. One portion was fired at one time and the balance at another. Further precaution was taken to distribute the specimens of any one sample throughout the setting whether in the kiln or door panel.

Fifteen bricks were tested at one time in the quenching test. Two thermocouples were placed near diagonally opposite corners in the door of the furnace and as close to the test bricks as possible. The closed end of the thermocouple protection tubes extended approximately $\frac{1}{2}$ in. beyond the face of the test bricks which was exposed to the furnace temperature. The temperature near the test bricks was controlled by means of these thermocouples. The location of the individual bricks in the door of the furnace was changed after each quenching.

TABLE II.—AVERAGE NUMBER OF HEATING AND QUENCHING CYCLES TO CAUSE FAILURE OF FIRE-CLAY BRICK IN THERMAL SPALLING TEST.

**Brick manufactured by three different processes and tested by five different methods for resistance to thermal spalling.**

| Process of Manufacture | Number of Brands | Average Number of Cycles for Failure | | | | |
|---|---|---|---|---|---|---|
| | | Method 1[a] | Method 2[a] | Method 3[a] | Method 4[a] | Method 5[a] |
| **Semi-dry press** | 10 | 22 | 33 | 25 | 27 | 7 |
| **Hand-made** | 7 | 10 | over 42 | 36 | 35 | 5 |
| **Stiff-mud** | 3 | 7 | 15 | 19 | 28 | 4 |

[a] **The following are the salient points of difference between the five methods:**

| Method | Preheating at 1400° C. | Quenched from, deg. Cent. | Depth of Water, in. | Amount of Brick Spalled | Method of Removal of Loose Pieces |
|---|---|---|---|---|---|
| **No. 1** | **Entire Brick** | **850** | **4** | **Entire end** | **Manual** |
| **No. 2** | None | **850** | **4** | **Entire end** | **Manual** |
| **No. 3** | **End only** | **850** | **2** | **Entire end** | **Gravity-manual** |
| **No. 4** | None | **1350** | **2** | **20 per cent** | **Gravity** |
| **No. 5** | None | **1350** | **2** | **Entire end** | **Manual** |

## RESULTS AND DISCUSSION

The type of failure and the amount of loss sustained when open- and close-textured bricks were subjected to the several different spalling tests are illustrated in Fig. 1. The results obtained by the several methods for testing the resistance of fire-clay brick to thermal spalling are given in Table I. The method of manufacture and geographical source of each brand of bricks is also given.

Based on the group averages of heating and cooling cycles to cause spalling, it is found that the effect of a preliminary heating of the bricks at 1400° C., whether the entire brick or only one end is so treated, is most harmful to those from Pennsylvania, and next in order are Kentucky, Missouri and California bricks. However, the results on the California bricks are based on only two brands and the comparison cannot be considered altogether equitable. This effect of preheating is evidently caused by certain characteristics of the clays and cannot be interpreted as reflecting on the relative qualities of the brick in service.

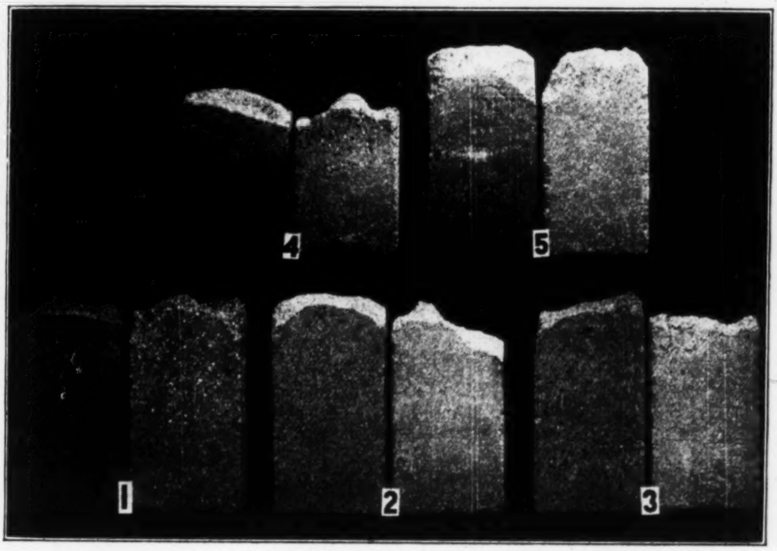

FIG. 1.—Showing the Effect of Thermal Shock on Bricks of Open and Dense Structure When Tested by Five Different Methods.

The dense brick is to the left of each number; the numbers refer to the method of testing.

The values on the average weight loss as given in Table I show very little difference when the bricks are tested according to methods 1, 2, 3 and 5, but the loss in the case of method 4 is practically double that obtained with any of the other four methods of tests. This is well illustrated in Fig. 1. However, the requirements of method 4 account for the larger loss.

The percentage mean deviation from the average in the number of cycles to cause failure of the bricks is practically the same for the A.S.T.M. method and that of the Federal Specifications Board and is greater for these two methods than for the other three methods of testing.

The results obtained when the bricks are tested according to the Federal, the A.S.T.M. and one of the modified methods (No. 3) of thermal spalling tests are shown in Fig. 2. The purpose of the curves is to compare graphically the results obtained when testing bricks by the two present standards. The third curve is added for purposes of comparison. A visual inspection of the preheated bricks represented by letters on the right one-third of the graph shows that they are of a decidedly denser structure than those represented by the letters on the left of the graph.

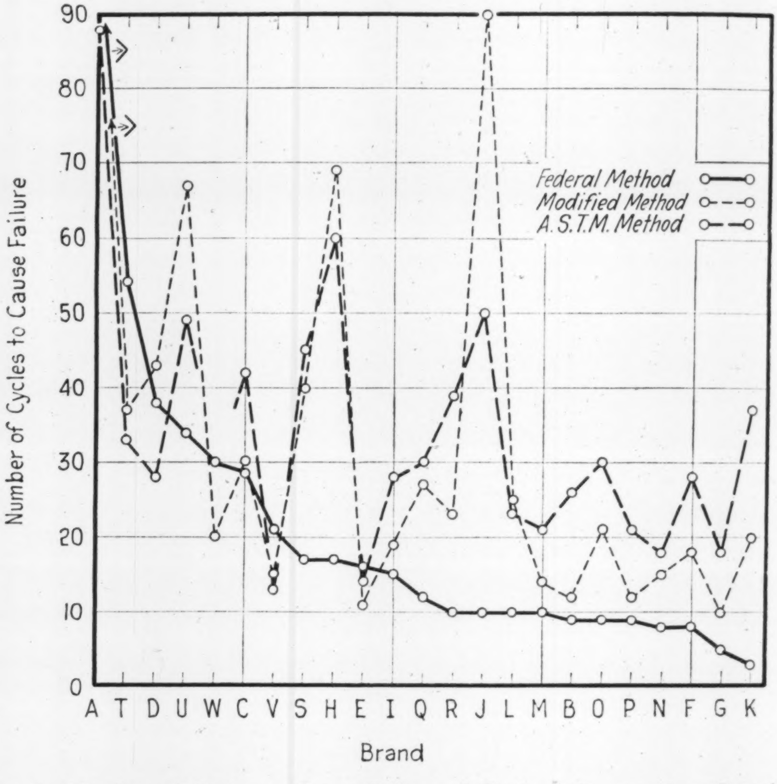

FIG. 2.—Number of Cycles to Cause Failure of Each of Twenty-three Brands of Fire Brick When Tested for Resistance to Thermal Spalling by Three Different Methods.

The results of the spalling tests obtained by methods 1, 2, 3, 4 and 5 on twenty different brands of bricks representing three different processes of manufacture, are further summarized in Table II. (The data do not include values for brands A, U and W.) It may be seen from Table II that:

1. When testing without preheating, a greater number of cycles is necessary to cause spalling than when preheating is used. The hand-made bricks are favored most since the increase in number of cycles to cause spalling in that type of brick greatly exceeds that obtained for bricks made by either the semi-dry press or stiff-mud process.

2. The semi-dry press bricks are the least sensitive to method of test, while the stiff-mud are the most sensitive. This point is brought out more clearly if only values for methods 1 to 4 inclusive are considered.

The following are the principal advantages and disadvantages of the six test methods as brought out during the course of the study and by the data summarized in Tables I and II.

*Method 1.* (Federal—Quenched from 850° C. in 4 in. of water; entire brick preheated).—

(*a*) Gave a lower number of cycles to cause failure than all other methods with the exception of method 5.

(*b*) Indicates bricks manufactured by the dry-press process to be the most resistant to spalling.

(*c*) The manual removal of the loosened pieces is unsatisfactory because of the personal element involved.

(*d*) Percentage mean deviation from the average number of cycles to cause failure is relatively high.

(*e*) Preheating entire brick at 1400° C. does not occur in service installations, nor is it just to the manufacturer whose bricks are "hard burned" in comparison with one whose bricks are "soft burned."

(*f*) The "spread" or difference in the number of cycles to cause failure used as a means of differentiation for bricks representing individual brands, as well as the three processes of manufacture, is relatively good.

*Method 2* (Quenched from 850° C. in 4 in. of water; no preheating).—

(*a*) Greater number of cycles obtained in this test than in any other.

(*b*) Indicates bricks made by the hand-made process to be the most resistant to spalling.

(*c*) The manual removal of loosened pieces is unsatisfactory.

(*d*) Percentage mean deviation from the average number of cycles to cause failure is fairly low.

(*e*) Because of large number of cycles obtained, test is rather prolonged.

(*f*) The difference or "spread" in number of cycles to cause failure of the bricks manufactured by the three different processes as well as between individual brands is very good.

(*g*) The bricks during this test are subjected to very little if any change in structure caused by vitrification.

*Method 3* (Quenched from 850° C. in 2 in. of water; one end preheated).—

(*a*) Indicates bricks made by the hand-made process to be the most resistant to spalling.

(*b*) Method of removing loose pieces of brick an improvement over methods 1, 2 and 5, because the personal factor is reduced to a minimum.

(*c*) The difference in number of cycles to cause failure for the bricks manufactured by the three processes as well as for individual brands is large enough to define differences in the spalling properties of the bricks.

(*d*) A preliminary heating of one end of the brick simulates treatment received in service installations.

(*e*) Percentage mean deviation from the average number of cycles to cause failure is intermediate between highest and lowest values obtained in the other tests.

*Method 4* (A.S.T.M.—Quenched from 1350° C. in 2 in. of water; no preheating).—

(*a*) Indicates bricks made by the hand-made process to be the most resistant to spalling.

(*b*) Removal of loose pieces by gravity is fairly satisfactory because it more nearly duplicates service conditions.

(*c*) The "spread" in number of cycles to cause failure between the three methods of manufacture is relatively small and there is no significant difference in the case of bricks made by the stiff-mud and semi-dry press processes.

(*d*) The percentage mean deviation from the average in number of cycles to cause failure is rather high and is approximately the same as obtained with method 1.

(*e*) Because of the high percentage of loss in weight of the brick, required by the method, the test is somewhat prolonged.

(*f*) Slow vitrification of one end of the brick is similar to service conditions.

*Method 5* (Quenched from 1350° C. in 2 in. of water; no preheating).—

(*a*) Smallest number of cycles to cause failure, making it impossible to differentiate satisfactorily between different brands of brick.

(*b*) Manual method for removal of loose pieces of brick unsatisfactory because of personal element involved.

*Method* 6 (Not considered because only seven brands were tested).—

Methods 1 (Federal) and 5 indicate that brick manufactured by the semi-dry press process are more resistant to thermal spalling than those manufactured by the hand-made process. Service conditions in general indicate the reverse should be the case.

The order of resistance of the three types of bricks to spalling shown by the results obtained by methods 2 and 3 is the same as general opinion rates them in service.

The personal factor involved in removing the loose pieces of brick as practiced in methods 1, 2, and 5, may cause decided errors in any values obtained.

## CONCLUSIONS

Results obtained in the Bureau of Standards investigation of fire-clay refractories[1] during the past several years as well as those of the present study indicate that some change in the present Federal method of determining the thermal spalling resistance of fire-clay brick is desirable. Results obtained in the former investigation show that the most advisable changes are to modify the preliminary heating procedure and the method of removing the loosened pieces. Such heating is apparently beneficial to so-called "soft burned" brick and detrimental to "hard burned" brick so far as test results are concerned. However, because of the time consumed in making a test on the brick without a preliminary heating (method 2) and because vitrification of one end of the bricks takes place in service, it would seem logical to cause similar vitrification in the test brick. This resulted in the development of method 3. The quenching was made in 2 in. of water instead of 4 in. because of the limited extent of the partly vitrified portion. In addition, under operating conditions, the face of the brick exposed directly to the furnace temperature would receive the greatest thermal shock. The gravity-manual method for removing loosened pieces was substituted for the manual method because it eliminated to a great extent the personal factor involved.

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[1] "Progress Report on Investigation of Fire-Clay Bricks and the Clays Used in Their Preparation," Bureau of Standards *Research Paper No. 114.*

## DISCUSSION

MR. S. M. PHELPS[1] (*presented in written form*).—Mr. Heindl's work on the water-dip spalling test supplies information which has long been needed in the way of comparing two widely-used test methods. These data and his discussion of each test method leave little doubt as to the merit and shortcomings of the individual tests.

It is, of course, difficult to make direct comparisons between life in service and the test data obtained from tests which employ principles so different from the actual conditions in service. The important point left unsettled by this paper is whether or not there is sufficient merit in test method 3 over the A.S.T.M. method (method 4) to warrant adoption of the former. It would seem that, in view of the large amount of data which has been collected by means of the A.S.T.M. method on practically every brand of brick, any method which should supersede it should be an outstanding one.

The data obtained show clearly that the present Federal Specifications Board procedure could easily be improved upon. If this should be done, it is hoped that the method adopted will be a standardized one so as to avoid the necessity of obtaining two sets of spalling test data and the interpretation of these in terms of service. Obviously, this would simplify the writing of the specifications, and reduce the cost of testing brick to comply with them.

MESSRS. F. A. HARVEY[2] AND J. S. McDOWELL[3] (*presented in written form*).—The paper by Mr. Heindl contains much valuable information worked out in his usual careful and systematic manner. It is particularly advantageous that the paper be discussed before this Society since the question arises as to whether the Society's Committee C-8 on Refractories should continue its present Tentative Method of Test for Resistance of Fire-Clay Brick to Spalling Action (C 38 - 27 T) or modify it to conform to one of the other methods studied by Mr. Heindl.

The past history of the A.S.T.M. water-dip spalling test is of considerable interest. Prior to 1921, a subcommittee of Committee C-8 had been working on a water-dip spalling test. The method most favored at that time was practically the present A.S.T.M.

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[2] Harbison-Walker Refractories Co., Pittsburgh, Pa.

[3] Director of Research, Harbison-Walker Refractories Co., Pittsburgh, Pa.

tentative method. The subcommittee, however, was not ready to propose a method due to the fact that tests made on a few well-known brands failed to classify them in the order in which they were known to resist spalling in service. Raymond Howe, with Ferguson, made some tests late in 1920 by preheating the brick for five hours at 1400° C. and then carrying out the water-dip spalling test. The results of this study were published in the *Journal* of the American Ceramic Society.[1] The study was carried out on only seven brands of brick but indicated that the test classified the brands approximately in the order of their resistance to spalling in service. On the basis of the results, the subcommittee recommended and Committee C-8 adopted a tentative spalling test which used the five-hour preheating at 1400° C. and subsequent water-dip test.

During the next five years, a large amount of data both on service and results in the test was accumulated on many brands of brick which were not included in the original study. These more comprehensive data showed that the original study was carried out on too small a number of brands, and disproved the conclusion that the results of the test could be correlated with service. Even after the first year it was evident that the test was faulty, but as no better test was devised, it was continued from year to year as tentative. By 1926 the mass of adverse evidence was so great that a modification of the test was unavoidable. Among other factors it had been clearly established that the burning treatment given the brick during manufacture was of importance in influencing its resistance to spalling in service. The tentative test, by its five-hour preheating treatment, nullified the effect of the burning treatment during manufacture. The 1926 modification attempted to remedy this situation by eliminating the five-hour preheating treatment.

During the period 1921 to 1926, some fundamental conceptions of the mechanism of spalling had gradually developed. These conceptions are briefly summarized in the definition for spalling now published by this Society in the Standard Definitions of Terms Relating to Refractories (C 71 – 31).[2] This fundamental work showed that the mechanism of spalling under service conditions involved many factors, of which resistance to thermal shock is only one. Thermal spalling seldom, if ever, occurs apart from one or more of the other types. Contamination by slags or fluxes, pinching, shrinkage or vitrification, alone or in combination, are usually present. A given series of brands does not show the same order of merit under different service conditions where spalling is severe. For example, if a series is arranged in order of merit for boiler arches, this same series will take a different order for malleable furnace bungs. Even in the same general type of service, as in boiler furnaces, but in different plants, the operating conditions will differ, and the brick which is best in plant A will not necessarily be the best in plant B even though the main cause of failure is spalling. Therefore, no single test could predict service under all possible conditions. Confirming this view, the mass of evidence shows that no one of the water-dip tests so far devised and studied can be used to classify brands in the order in which they will resist the various spalling influences in service. An attempt was made in 1927 to meet this situation and limit the application of the test to thermal shock, by changing the title of the tentative method to "Resistance of Fire-Clay Brick to Thermal Spalling Action."

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[1] *Journal*, Am. Ceramic Soc., Vol. 4, p. 32 (1921).

[2] 1931 Supplement to Book of A.S.T.M. Standards, p. 72.

A different type of spalling test had been studied at Mellon Institute for the past three or four years. The method is described in the *Journal* of the American Ceramic Society.[1] In this method, the brick are built into a panel and thus heated and cooled from one face only. Method 3 of Mr. Heindl's paper possesses this same excellent feature. Such procedure approaches conditions of service. Mr. Heindl's method possesses some advantages from the standpoint of manipulation. It can be carried out without special equipment, and the cost per test is lower than the Mellon Institute method. In the final quenching, however, the Mellon Institute method possesses the advantage that the brick are still built in a panel, while with Mr. Heindl's water-dip quenching, the brick are cooled for 2 in. on the sides as well as on the end.

It should be emphasized again that no one of the spalling tests so far studied can be used as a suitability test. No one of them classifies all brick in the order in which they resist spalling under all the complicated conditions which exist in service. The suitability of a given brand for a particular service condition must be established by other means such as by trial in the furnace under normal operating conditions. Once the suitability has been established either the present A.S.T.M. tentative test or one of those covered in Mr. Heindl's paper may be used by either user or producer, as a control test, to make certain that uniform quality is being maintained.

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[1] *Journal*, Am. Ceramic Soc., Vol. 14, p. 389 (1931).

# SYMPOSIUM ON WEATHERING CHARACTERISTICS OF MASONRY MATERIALS

## INTRODUCTION

By D. W. Kessler[1]

In sponsoring this symposium the Coordinating Committee on Weathering Characteristics of Masonry Materials felt considerable hesitancy in approaching a subject in which information is so incomplete and inconclusive. Although considerable research has been carried out during the last half century on weathering qualities of various masonry materials, a review of this work will show that the materials and methods involved were so varied that a correlation of the results is very difficult. This assemblage of papers has been presented more for the purpose of showing the present status of the subject and serving as a starting point for more comprehensive and systematic research rather than the presentation of definite conclusions. The importance of the subject is becoming more evident every day and it is hoped that our efforts will serve the purpose of stimulating more interest and study along these lines.

It would appear that we have all made the common error of assuming that masonry weathering is a simple process. When one considers the great variety of minerals and complex chemical compositions entering into the field of masonry as well as the multiplicity of conditions to which the materials are exposed, he may better appreciate the intricate nature of weathering effects. As it is learned that certain combinations of minerals or substances in one material are non-resistant, it may also be learned that certain combinations of masonry units are less resistant than where the same materials are employed separately.

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[1] Civil Engineer, U. S. Bureau of Standards, Washington, D. C.; Chairman, Coordinating Committee on Weathering Characteristics of Masonry Materials.

# ECONOMIC ASPECTS OF MASONRY DECAY FROM WEATHERING

By H. S. Brightly[1]

## Introduction

The purpose of this paper is primarily to point out the serious economic aspects of masonry decay and its generally preventable character, preventable except, of course, in the case of the use of decidedly inferior materials incapable of resisting exposure in the localities in which they are employed. A considerable portion of the paper will be devoted to details of construction that are now employed or can be employed with various forms of material to prevent decay from weathering.

The subject of "weathering" and "weathering-decay" is certainly not a new one, as by force of circumstances it has been known and has through necessity engaged the attention of builders from time immemorial. The decay of masonry has always been a serious problem in building construction, quite regardless of the fact that masonry, generally speaking, when built of suitably selected materials, is considered the most nearly permanent and enduring form of construction.

While the term "permanence" is very loosely applied in modern times, it is generally recognized by technically trained men that this is a relative term and in the last analysis there is no such thing as "permanence;" that while certain structural materials, notably well-burned clay and the more durable and lasting of rock formations, will resist the elements effectively for many centuries of time, these must in time give way to the disintegrating effects of continued exposure over a long period of time; even the most stable of granites and other durable rocks in time split up and gradually separate into their constituent mineral elements.

Thus, in considering even the most permanent of building materials, it is essential that the users of these in building or engineering construction give careful consideration to all of the conditions of exposure and, by the details of construction employed, provide as far as economically practical against all unduly severe and untoward agencies of exposure that may act upon these materials and tend to cause their more rapid weathering or decay under the conditions imposed by the particular form of usage. Here, also, it appears appropriate to comment that the form of usage is another factor, that is, it makes some difference whether the particular material is to be used in small blocks or units, whether it is to be used in blocks having considerable thickness, or in comparatively thin slabs, also the manner in which it is to be built into or secured to adjoining masonry, and whether the conditions of usage are such that will permit the normal amount of expansion and contraction from temperature change without causing excessive stress to be exerted upon the material itself. By that, reference is meant both to the manner in which the material is confined in its position in the building or may be stressed by the expansion movements of other adjoining materials or structural members. A notable example of this is the manner in which a comparatively thin veneer facing of masonry may be affected by movements that take place in the structural members of a tall building, having a structural steel or a reinforced concrete frame structure.

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[1] Secretary-Director, Building Stone Association of Indiana, Inc., Bloomington, Ind.

The moisture content of masonry materials and the effects of this on the life of the structure has only during comparatively recent years been given the serious consideration that this seems to deserve, as this important factor in the causation of masonry decay may vary, not only from season to season but from week to week, possibly even from day to day, and vary also quite markedly in different parts of the structure, with its swelling and dissolving action always at work. It is, therefore, thought that all of these factors should receive careful consideration in the design of structures as a means of preventing the failure from causes which, though looked upon as physical forces rather than weathering, may contribute their quota to an accelerated decay, or cause a more rapid than normal "weathering" of masonry materials.

## DECAY AND WEATHERING

Before proceeding with a discussion of masonry materials, the manner in which they are affected by destructive agencies, and the economic aspects of these phenomena, it is thought well to define what is meant by the terms "weathering" and "decay." For the purpose of this paper the term "weathering" will be employed as the gradual wearing away of any building material that is caused by normal exposure to the atmospheric agencies, sun, wind, rain, and temperature change, including snow and frost action. By "decay" is meant the destruction or disintegration of the material, either wholly or in part to a considerable depth, from any cause. The term "decay" thus is used to define the more or less gradual loss of structural integrity caused by other than a shattering stress, either from excessive loading, from shock, or from fatigue due either to the reversal of stress beyond the elastic capacity of the particular material, or continuous loading stress beyond the safe structural capacity of the particular material. While the seriousness of any marked decay of masonry from weathering is fully appreciated by engineers and others having to do with building structures in a professional capacity and has engaged their attention for a long time and has been made the subject of many worthy technical researches, it seems doubtful to the author if the real economic aspect of such decay has ever been appreciated by the building public, or by those having to do with the making or handling of investments in building projects, of both public and private nature. The building owner or investor usually learns to his sorrow all too late of the difficulties that so often could be prevented by a moderate increase in the original investment—serious trouble which must then be corrected at considerable expense in order to maintain the integrity of the building, and in the case of a commercial enterprise the soundness of the investment.

## MATERIALS

This paper, being broadly titled to cover all forms of masonry, will discuss the weathering effects on the various types and kinds of building stone, burned-clay products and the cast or molded materials composed of cement and aggregate mixtures. While there are certain other materials belonging to the masonry class that may be used for exterior building purposes, such, for instance, as stucco, these can hardly be classed as masonry, but rather as applied finishes for masonry, and the weathering of such materials being a study in itself is, therefore, not embraced within the limits of this paper.

## MASONRY BOTH A STRUCTURAL AND ORNAMENTAL MATERIAL—THE BASIC IMPORTANCE OF ITS INTEGRITY TO THE VALUE OF THE STRUCTURE

Since masonry, as employed in building and engineering construction constitutes the enclosing and supporting members of the structure, the life of the structure and, consequently, its economic value, depends to a very large extent on the durability of this portion of the whole construction. The physical life of any building or other structure, barring destruction by fire and internal decay, generally speaking, is dependent on two things, the integrity of its supporting and enclosing walls and the integrity of its roof, or top cover paving. This seems so obvious and basic, it is astonishing that so many chances will be taken by designers both in the employment of materials and in working out the structural details of the masonry enclosures. It seems that designers are inclined to take too much for granted in the permanence of masonry materials and do not give sufficient consideration to the fact that while a specimen of burned clay or a rock mass may be practically indestructible and permanent when left to weather naturally or even under ordinary experimental tests in the laboratory, these same materials when built into the wall or other part of a structure, may be found infinitely less durable.

Builders have come to take too much for granted from masonry materials on the basis of laboratory tests and often use too little common sense in the employment of these materials in building construction. In consequence, numerous structures throughout the country are badly disfigured and in some instances the integrity of the walls more or less destroyed by efflorescence. In other instances portions of the work have been split and to some extent disintegrated by frost action. Generally speaking, repairs, to say the very least, are costly and sometimes almost impossible of execution without tearing down entire portions of the construction. Thus, the economic value of a building is to a large extent dependent upon the integrity of its masonry work and its roof construction, over the normal life period or usefulness of the structure. This, in the case of memorials and monumental buildings and engineering structures, may and should run into centuries of time.

Most architects are more or less familiar with the destructive effects of moisture in masonry, regardless of whether this penetration of moisture occurs from leaky roofs, gutters or parapets, or through the absorption of moisture by capillary action from the surrounding soil. Thus, in using masonry of any kind it is important that even this most permanent form of building be properly protected against moisture, as well as from any excessive weathering effects from other atmospheric agencies. In the employment of masonry this means not only the selection of the material itself, brick, stone, or other material, but also the manner of bedding or building and cementing the material used into the form and mass that is to constitute the wall, pier, or other part of the construction. The mortar used thus is an important factor and especially so in modern construction, where a comparatively thin wall section or envelope of masonry is used to enclose a structural frame, a wall that is more or less subjected to movements from sun action, moisture content, and temperature change. This binding medium of the masonry is often even more important than the material itself, since the integrity of the completed masonry is frequently but little more secure than the integrity of the material by which it is joined together in its structural form, as a wall, pier, or abuttment. This is especially true in the case of parapets, copings, balustrades and the exposed upper portions of any structure where moisture penetration at the point of connection with roofs and gutters or conductor drains, as well as from rain and melting snow, must be provided against.

## INADEQUATE PROTECTION AND COSTLY REPAIRS

When one reviews the costly troubles that have occurred through the improper use of some form of masonry, or its inadequate protection, one is surprised at the enormous waste that has been brought about either through improper specifications, or the inadequate enforcement of specification requirements. The following illustrations may be cited:

*Case No. 1*, a tall office building in a large eastern city. Due to the improper laying of the brick, insufficient mortar for properly bedding the brick, hasty work, careless workmanship, and, in consequence, a wall full of voids and open spaces (the brick itself being in every respect first class), the expenditure of something over $100,000 was made in an attempt to point up and waterproof the joints and prevent driving rains from penetrating the wall, causing dampness on the interior, ruining the decoration and finish of the interior offices.

*Case No. 2*, ecclesiastical building constructed of local stone with limestone trim. The walls were built "piece-meal" with inadequate protection, as funds were available, the masonry vaulting filled with concrete being subjected to melting snows through the winters of a number of years, resulting in efflorescence causing "wall cancer" that practically ruined portions of the elaborately carved limestone, necessitating repairs which if carried out would mean the replacing of entire portions of the construction at an expense that would run into many thousands of dollars—probably over fifty per cent of the entire original cost.

*Case No. 3*, a wrongly used and inadequately supported and protected burned-clay product, replacing granite in the upper portions of an important monumental building in a mid-western city. The penetration of moisture caused splitting and disintegration of the product, rusting of anchorage necessitating repairs that involved an expenditure of something over $200,000.

These more or less outstanding cases of the inadequate protection of proven materials which in laboratory tests would withstand every reasonable weathering test are cited simply to illustrate the outstanding economic importance of weathering decay from the standpoint of the building owner or investor.

## TYPES OF WEATHERING DECAY

In weathering, the building materials employed in the exterior of structures are affected by a number of different types of weather action tending to cause the wearing away or surface disintegration of the material. These can be enumerated as follows:

1. Frost action, due to the freezing of moisture that has been absorbed by the material itself or penetrated the mortar joints either from driving rains or a supersaturated atmospheric condition.

2. Chemical weathering, embracing the wearing away or etching action and eating away of the surface by the acids in rain water and humid atmospheres absorbed from coal smoke.

3. Efflorescence, or the exudation of soluble salts contained either in the masonry material itself or in the mortar materials employed in laying up the masonry, that have become dissolved by moisture finding its way into the body of the wall and later evaporating through the exterior surface. The penetration of moisture capable of causing this effect may be from various sources:

   (*a*) Absorption by capillary action from the soil surrounding the base of the building.

   (*b*) Direct penetration or percolation from rains and melting snows.

   (*c*) The capillary absorption of moisture bearing these salts from other materials built in contact with or forming part of the masonry.

   (*d*) The leaching of destructive acid solutions from soot or other accumulations on the flat surfaces and ledges or projecting parts of masonry.

4. Biological action occasioned by vegetable growths or nitrifying bacteria deposited on the face and in the surface pores of the material.

5. Stresses on exposed surfaces caused by the cycles of wetting and drying, heating and cooling, due both to sun and rain and the seasonal changes in atmospheric conditions.

Weathering, as it actually occurs, may, of course, be caused by any combination of several or all of these agencies tending to wear away the exposed surfaces. It is thought that sufficient attention has not been given to the combined effects of these various weathering agencies, and, in the design and detailing of the wall or other construction and in the care with which masonry is built, not enough definite provision made to nullify their intensity and effect. For real durability the joints, especially the joints of copings and horizontal surfaces, should be made lastingly tight and waterproof. Untold damage to the exterior of a building may be caused by slighting any of the necessary means of preventing the penetration of masonry by moisture—moisture, the arch enemy of masonry, quite regardless of the known durability of the particular kind of material employed.

The effect of frost action is customarily eliminated by the selection of a material that is sufficiently resistant to moisture penetration and capable of resisting frost action to an extent that gives it the required durability and consequent reasonable freedom from danger from that particular source of weather action, but even with such materials, completed masonry may be seriously affected if the joints, especially the vertical joints in horizontal and wash surfaces are not made lastingly water-tight.

The effect of chemical weathering cannot be avoided and, ordinarily, when the right kind of material has been used, is not serious. The chemical weathering of any material, however, especially under more rigorous atmospheric conditions, can often be substantially alleviated by the use of a suitable surface waterproofing treatment.

Efflorescence is the one most unsightly and inexcusable fault causing the disfigurement and surface disintegration of masonry materials. Immunity from that trouble can be obtained first, by the selection and use of materials that do not contain soluble salts, and, second, by building them into the structure with thoroughly filled, dense, reasonably non-absorbent and moisture-proof mortar joints.

The integrity of a mortar joint, however, is not always entirely dependent upon the material employed. In the modern tall skeleton frame building, in which the structural frame is subjected to certain movements by seasonal change in temperatures, it is impossible, even with the best mortars, to insure over any considerable period of time the tightness of all vertical joints. For that reason in this type of structure an elastic caulking compound is often considered necessary as a means of preventing the unsightly disfigurement by efflorescence. Instances of efflorescence have been noted where this has caused a veritable "wall-cancer," where it occurs in particularly aggravated form, eating away not only the surface but deeply into the body of the material.

The effect of biological action can generally be prevented by treating the exposed surfaces of masonry with a waterproofing or other solution that will cause the closing of the pores and result in a non-absorbent surface that does not form a suitable background for any growths of this kind.

The effects of temperature change in the material itself cannot be avoided. The material must, by virtue of its elasticity, be capable of absorbing such stresses as may be set up in it by changing temperatures and seasonal conditions, without any destructive shattering.

### ECONOMIC EFFECTS

While the economic phase of the subject has already been touched upon and while it is utterly impossible, within the limits of this paper, to go into any great detail on the economic effects of masonry decay, it can safely be stated that the burden of avoidable masonry decay on the owners of and investors in building property is an enormous one, measurable not in thousands but in millions of dollars annually, for entirely apart from the major repairs that are necessary in overcoming outstanding instances of excessive weathering or "masonry decay" affecting the exteriors of buildings, there is the continual expenditure for minor repairs of a routine nature in keeping the exteriors of buildings reasonably weatherproof and in a presentable condition from the standpoint of appearance.

It would indeed be a Herculean task to compile any statistics on this subject and separate the really uncalled-for maintenance expenditures from those that are normally necessary in properly designed and carefully constructed buildings. It can safely be said, however, that fully fifty per cent, if not more, of that huge annual expenditure could be avoided. Along with this we have the all-too-rapid depreciation in economic value of many buildings, due to the manner in which the appearance of their exteriors becomes affected by exposure to weather action, quite regardless of whether the structural integrity of the masonry itself has or has not deteriorated to any appreciable extent. In many instances unnecessary maintenance costs are entailed by small savings in the original expenditure, through omitting desirable specification requirements, which result later on in an expenditure, to overcome or correct the troubles for which these omissions are responsible, several times as great as the amount saved by these omissions. To explain what is meant just one illustration will be cited. The weatherproofing of vertical joints in stonework is omitted, by leaving off either the metal flashing or the elastic caulking compound filling of the joints, that would provide the required protection. Later on either one of these two expedients must be resorted to.

In the case of the caulking compounds, this means that the expenditure is not only the installation of that material, but also for the cutting out of old mortar, often an expensive operation, and for the cleaning off of streaked and disfigured wall surfaces, or, in the case of the metal flashing, the installation not only of the flashing required but a much more costly job due to this having to be provided for after the building is up and be joined to and made waterproof with roofing, gutter lining and other features of the construction already in place.

The correction of trouble due to faulty original construction of that kind is always more costly, sometimes several times the original cost of providing better details and the installation of materials that would afford the required protection.

Experienced architects and building owners who have made a careful survey of troubles of this kind will know what a heavy toll of maintenance expense is involved in overcoming preventable weathering, which, if allowed to go on, would seriously affect and impair the investment value of the buildings thus affected.

## SUMMARY

In summarizing, therefore, it seems important to point to moisture penetration as the big factor in considering the surface disintegration, or decay, of masonry materials, regardless of the exact source or character of this decay. And, in consequence of this, to point out the importance of taking every means of preventing the penetration of moisture into the body of masonry from any source whatsoever, quite regardless of the character and durability of material employed.

# THE PERFORMANCE OF CONCRETE IN EXPOSED STRUCTURES

By Ephrem Viens[1]

## Synopsis

From an inspection of natural stone and concrete in exposed structures and with a knowledge of weathering agents, it becomes apparent that the most desirable features of a building material, which is to be durable, are uniformity and homogeneity.

Portland cement is now a reliable material and much improved since first placed on the market but there is still considerable to be learned about it. The answers to some of the questions which could be asked about its characteristics might not prove of any great value in themselves but a more thorough understanding of its properties should make for the more intelligent use of concrete.

The first requirement of aggregates is that of soundness and at present there is a need for a rapid method of determining this property.

Uniformity and workability are absolutely necessary if durable concrete is to be obtained and these can be secured through care in preparing aggregates, proportioning, mixing, transporting and placing. Proper curing is also essential and the more careful choice and training of labor should be a help.

Architectural lines suitable for the use of concrete should be so chosen that the concrete may be well and efficiently placed and points of weakness such as sharp corners and undrained portions should be eliminated wherever possible.

Only a small percentage of the total quantity of concrete placed is deteriorating rapidly and cement and aggregates are seldom the cause, while carelessness and poor methods of application are usually to blame.

## Comparison of Natural Stone and Concrete

An inspection of stone in a natural cliff, a quarry face, or even a dressed block will reveal a lack of uniformity. In a rock face which has been exposed to the weather for many years, the effect of this feature is well brought out. The softer and less durable material is worn away, leaving the more durable stone projecting from the face. (See Fig. 1). The result is that through the lack of homogeneity the elements are permitted to act on a larger surface of the rock than if it all decayed at a uniform rate. In other words, the rapid weathering of the poorer material accelerates the wear on the better stone. Where concrete is not of uniform quality, similar results may be observed (Fig. 2).

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[1] Director, Testing Laboratories, Department of Public Works, Ottawa, Ont., Canada.

Nature has worked without attempting to produce durable material. By chance she has produced both good and bad and often in the same mass. (See Figs. 3 and 4.) Man, on the other hand, has always desired a durable concrete, but in his early works has also produced by chance both good and bad. (See Figs. 5 and 6.) From this experience he has learned many lessons and now if he applies the knowledge he has gained, he can be reasonably sure that his material will withstand the elements for many years. (See Fig. 7.)

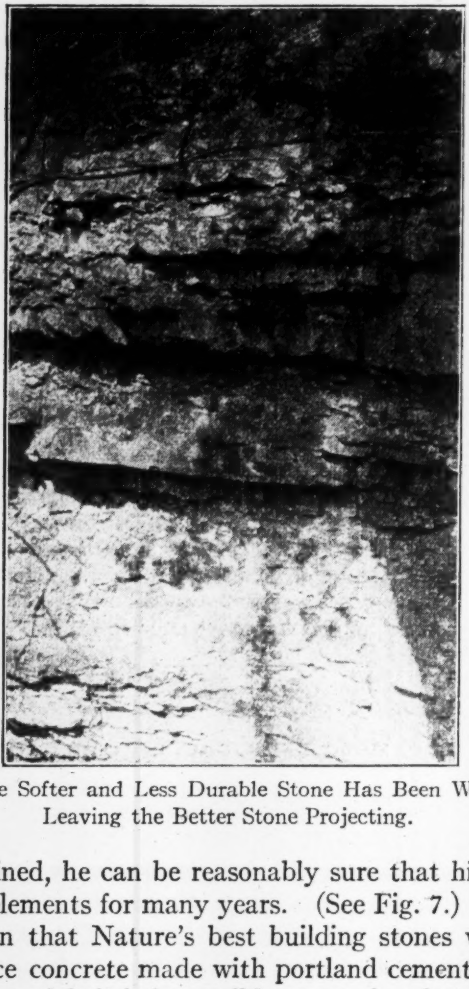

FIG. 1.—The Softer and Less Durable Stone Has Been Worn Away, Leaving the Better Stone Projecting.

It is known that Nature's best building stones will endure for centuries. Since concrete made with portland cement is a relatively new building material, it is impossible to say just how long it should be expected to endure. We do know, however, that some of the older structures have been in service for many years and are still performing satisfactorily. We know that the agents contributing to the destruction of building materials are wind, water, temperature changes, crystallization of salts and chemical reactions. These agents acting individually and collectively will, in time, destroy the best of materials. To resist most successfully nature's agents of destruction, an ideal material should be homogeneous and impervious. If a material possesses these characteristics, water, which is necessary for chemical reaction, dissolution and the formation of salt and ice crystals, will be restricted to the surface and temperature variations will produce stresses of a more uniform nature.

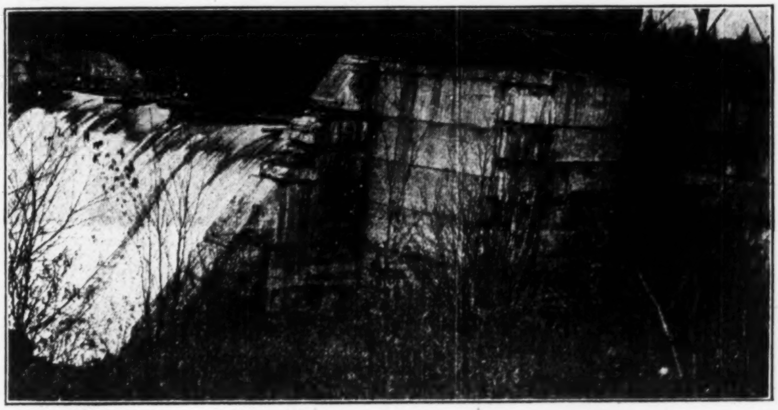

FIG. 2.—A Structure in Which the Concrete Has Been Eroded in a Similar Manner to the Stone in Fig. 1.

Due to segregation, the concrete is not of uniform quality, the fine and light material having run into the corners of the form.

From past experience man has learned how to make concrete with characteristics approaching those of an ideal material and with care should be able to produce concrete which will endure for hundreds of years.[1]

## CEMENT AND SOME OF ITS PROBLEMS

To form an appreciation of what should reasonably be expected of concrete and how it can be obtained, one must properly realize the nature of the materials of which it is composed. From the time portland cement was first made to the present day, its process of manufacture has been continually improved, making it a highly dependable material procurable in all parts of the world. However, in spite of the study which has been devoted to this material, there are a great many things which are not yet understood. Concrete and hardened cement paste will be to some degree porous, owing to the very nature of cement. This is assured from the fact that more water is required for mixing than is used in the hydration of the cement. We know, however, that if the ratio of water to cement is kept sufficiently low, the pores will be small enough to insure a more or less water-tight concrete.

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[1] C. H. Scholer, "The Durability of Concrete," *Proceedings*, Highway Research Board, Vol. 10, p. 132 (1930).

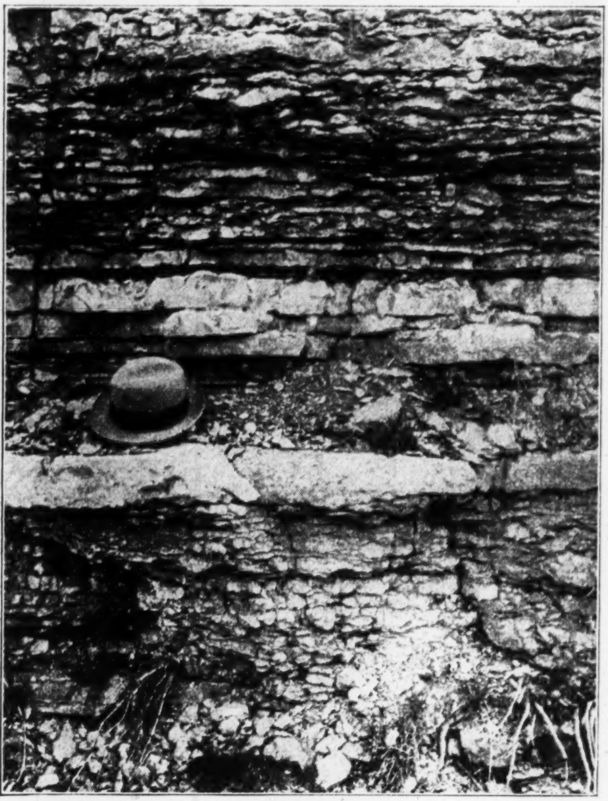

FIG. 3.—A Cliff Composed of Good and Poor Stone.

Many interesting questions can be asked about cement to which there are no answers at present. It is known that different brands of cement offer different resistance to the action of alkaline solutions (Fig. 8).[1,2] Does it follow that different brands also have different

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[1] T. Thorvaldson, "Studies on the Action of Sulfates on Portland Cement," National Research Council of Canada.

[2] D. G. Miller, "Resistance of Portland-Cement Concrete to the Action of Sulfate Waters as Influenced by the Cement," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 448 (1928).

durability factors when exposed to normal weathering conditions and if so, how important are these differences? Unfortunately, tests performed to determine the relative resistances of various brands of cement to deteriorating agents other than to alkaline solutions have not advanced as far as those on alkaline solutions alone. With improved processes of manufacture, the fineness of cement has been increased. Owing to this it has been found that structural strength requirements can be met with smaller quantities of cement than were necessary a few years ago. Crushing strength is used to a large extent to measure the quality of concrete, but it is doubtful if a concrete containing only sufficient cement to give a strength as required on ordinary work will contain enough to secure the durability desired of it. Studies on the fineness of cement show that to merely state a certain percentage of a sample passes a No. 200 sieve does not tell very much about the actual fineness.

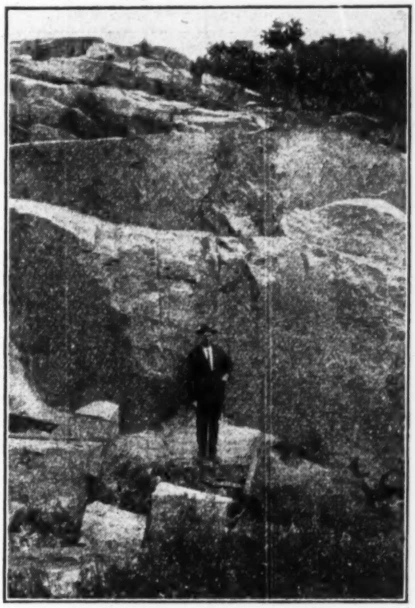

FIG. 4.—A Quarry Face of Good Stone.

Another interesting question is, to what extent do the physical properties of the raw materials enter into the properties of the finished product? It is known for instance, that in the case of hydrated lime a consideration of the chemical properties of the raw material and the process of manufacture are insufficient to explain the difference with respect to the plastic properties of the lime. Might this not also be true of some of the characteristics of various brands of portland cement, that is, that some of the physical properties may be attributed to the raw material from which they are made and hence cause a variation in their resistance to alkali and weather attack. If not due to the physical properties, then might it not be due to the slight difference in their chemical composition? Having discovered by test one brand to withstand alkali solution better than others, were the weaker brands in this respect brought to the same chemical compositions as the one which showed the best resistance, would they acquire the same resistance? If correcting the chemical compositions of these brands does not improve them, then it would appear that their performance is due to physical properties of the raw materials. The answers to some of the questions which could be asked about cement might not prove of any great value in themselves, but any knowledge that will facilitate the more intelligent use of cement or improve its qualities and those of concrete will warrant much study.

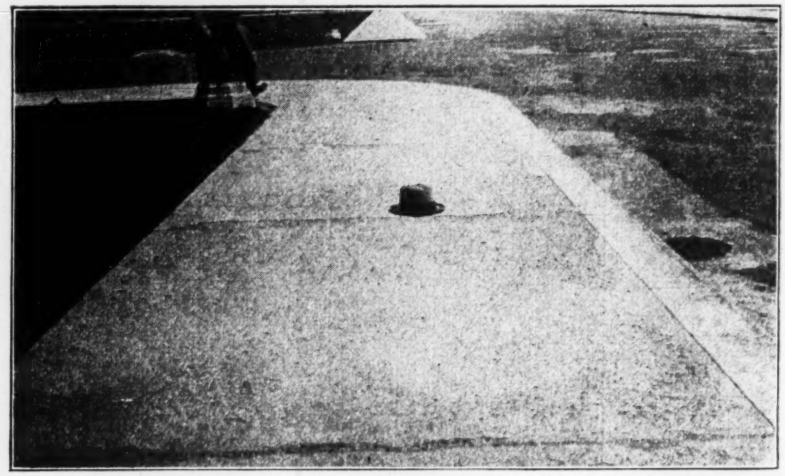

FIG. 5.—A Concrete Sidewalk After One Winter's Exposure.

This piece of work was done without any regard for control of the mix or good placing methods.

## AGGREGATES AND SOME OF THEIR PROBLEMS

The preparation of aggregates for use in concrete is becoming more and more a scientific problem. In many places it is developing into an important industry where excavating, crushing, screening and washing are carried out in a permanent and well-designed plant. In such places the choice of aggregates is a more simple problem than where the engineer in the field, often with limited time and facilities at his disposal, must choose between a few materials or often use the only one available within a reasonable distance.

The first requirement of an aggregate for use in concrete is no doubt that of soundness. Experience has shown that a visual inspection of a material is often insufficient to determine this factor as defects are sometimes of such a nature that they do not become apparent until the stone has been exposed to the weather for some time. An instance might be cited of a sandstone which, when set in place, showed no defects. A few years later, different strata were deteriorating at different rates, causing dark and light streaks in the stone, the light parts, being those where the cementing material is weaker, and where erosion takes place rapidly enough to prevent discoloring. When

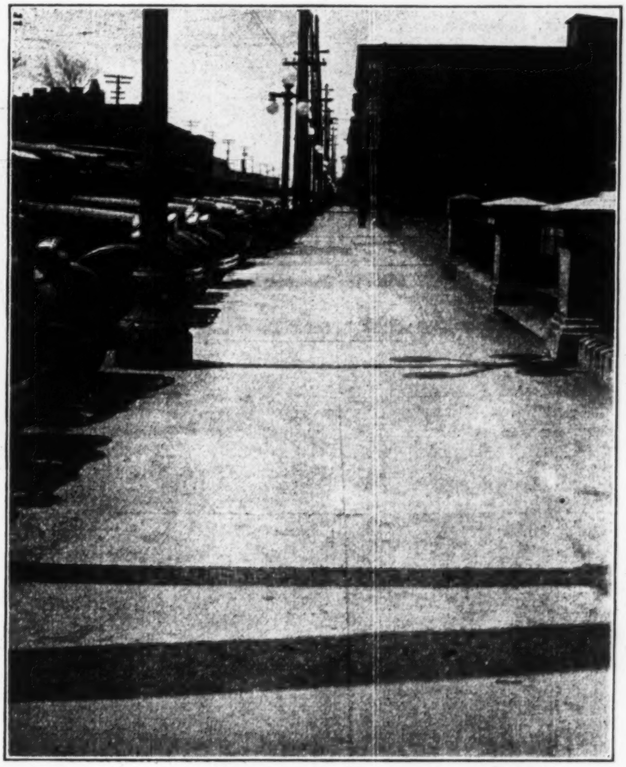

FIG. 6.—A Concrete Sidewalk Which Has Been in Service for over 40 Years.

It was built without present day knowledge of concrete and is still in perfect condition. The dark marks in the foreground are blotches of asphalt.

crushed stone is used however, an inspection of outcrops and exposed quarry faces, along with a knowledge of the time of exposure will often prove of great value, while freezing-and-thawing tests tell much more, but of course require considerable time. The sodium sulfate soundness test which is being developed at present may prove very valuable in this respect, when it is thoroughly understood. In the case of fine aggregate, it is generally conceded that it should contain very little material passing the No. 100 sieve. The exact nature of the ill effect of this material is not thoroughly understood and some studies in this direction would prove most interesting.

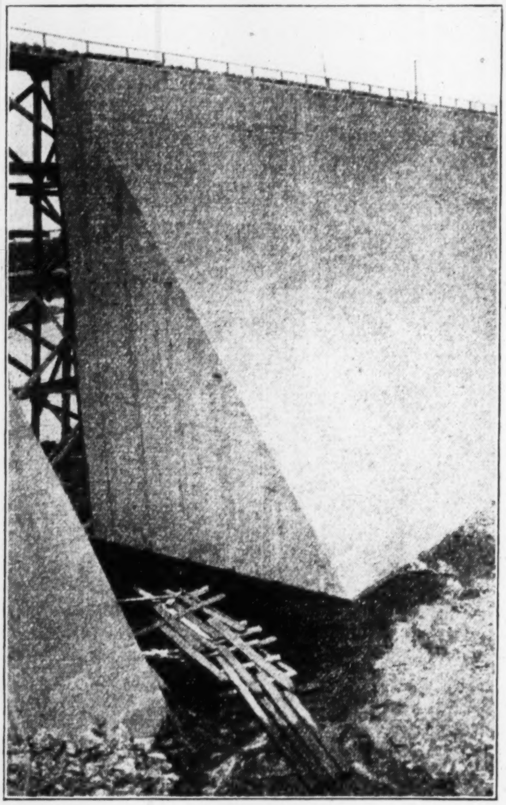

FIG. 7.—A Concrete Dam, Made Under Controlled Methods, Which is Expected to Endure for Many Years.

The section of the dam shown is from 60 to 70 ft. high and was constructed in one continuous operation, thus eliminating any horizontal joints or lift planes.

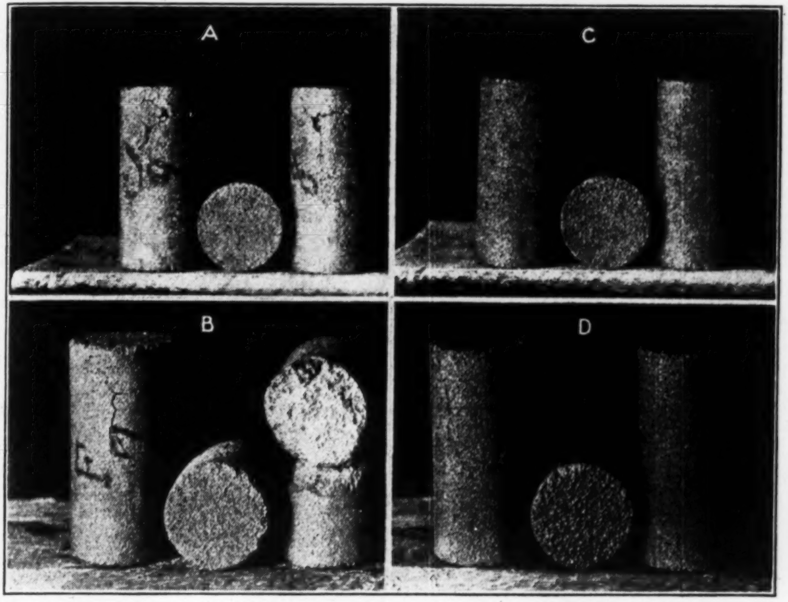

FIG. 8.—Four Groups of 2 by 4-in., 1:5 Standard Sand Mortar Cylinders Made from Four Brands of Portland Cement, A, B, C, and D.

**After curing for 1 day in moist air closet, 8 days in water and 3 days in air, the specimens were placed in a solution of sodium sulfate. The specimens were considered to have failed when they showed swelling and cracking and could be broken by twisting with the hands.**

**Cement A, failed at 56 days.**
**Cement B, sound at 56 days, failed at 65 days.**
**Cement C, failed at 107 days.**
**Cement D, sound at 107 days.**

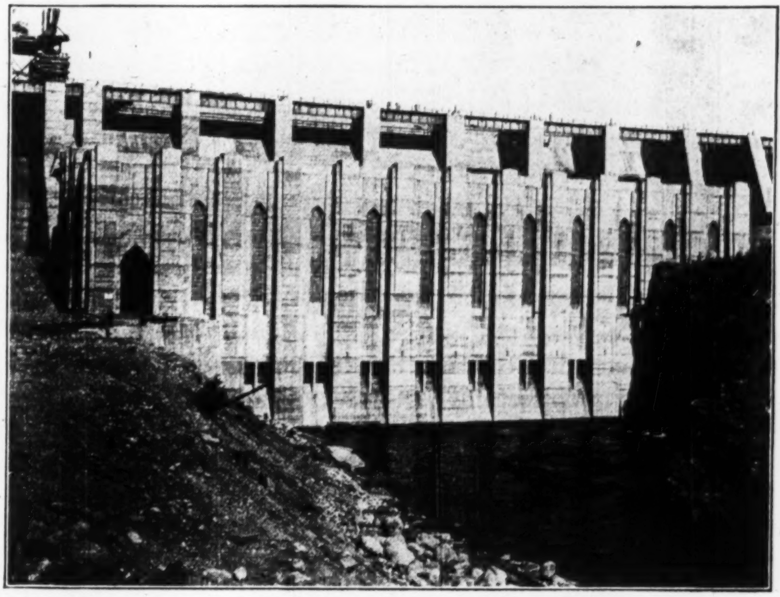

FIG. 9.—A Dam in Which Concrete Has Been Used to Good Advantage.

## CONSIDERATIONS IN SECURING DURABLE CONCRETE

### *Design of Structures:*

Many considerations are necessary in the building of durable concrete structures and these will require the cooperation of everyone who is in any way connected with them. Care must be taken in the designing office to see that the details as well as the general lines of the structure are such that concrete may be placed with reasonable efficiency and that architectural features are suitable for this material. (See Fig. 9.)

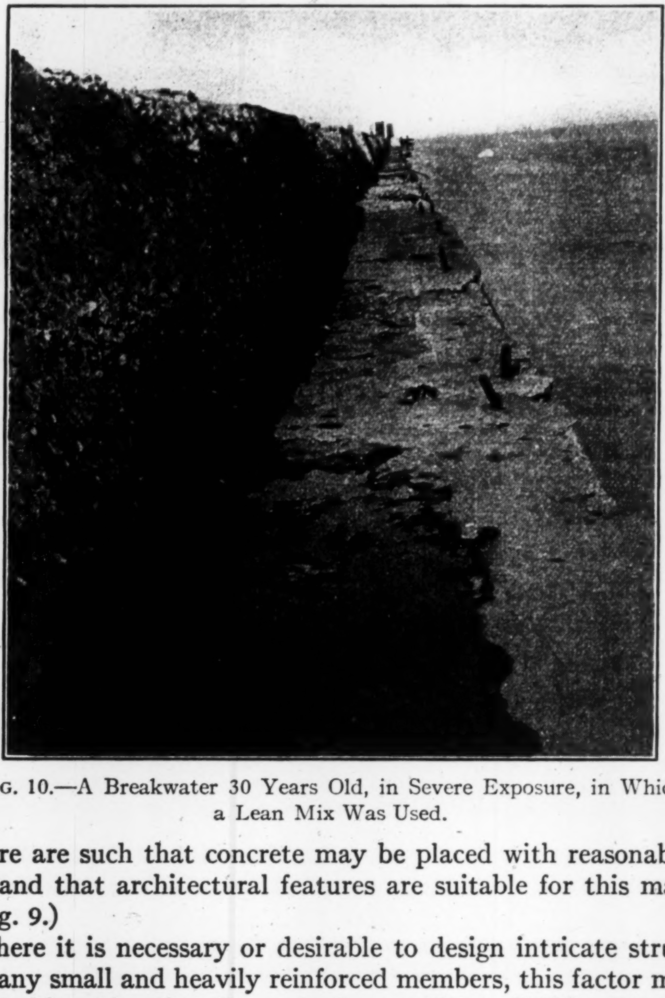

FIG. 10.—A Breakwater 30 Years Old, in Severe Exposure, in Which a Lean Mix Was Used.

Where it is necessary or desirable to design intricate structures with many small and heavily reinforced members, this factor must be considered in estimating the costs of concreting. If this precaution is not taken the construction costs will either exceed estimated costs or a poor piece of work will result. On the top and side of a structure exposed to the direct radiation of the sun, the concrete near the surface undergoes greater temperature changes than the interior of the mass. The result in some instances is that, after a very short time, the concrete near the contraction joint chips off leaving an unsightly "V" joint. This is probably not serious from a structural point of view but mars the appearance of the structures and might be overcome in some simple manner such as building with a "V" joint that would remain in good condition. It should not be considered necessary to follow old architectural lines when designing, but rather to develop new lines more suitable for concrete. Unless the most highly skilled workmen are employed and the greatest care taken in construction, many intricate decorations and pieces of fancy work will loose their decorative value after a very few years. But where old sharp lines of decorative effect must be followed, then care must be exercised that the concrete or mortar going into those parts be of the highest quality. Too often, indeed, only slushy mortar and sometimes laitance reaches these parts, hence one cause of their early deterioration. In many instances, sharp corners are not necessary and should then be eliminated since when present they are points of weakness which are readily broken off and attacked by weather.

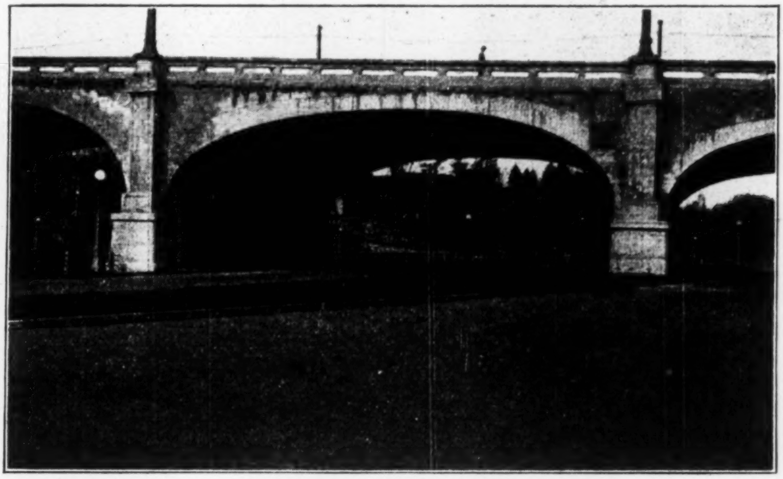

FIG. 11.—A Structure Built in 1912 on Which Approximately One-half the Original Cost Has Been Spent for Repairs.

**The result is an ugly structure which will have to be replaced long before it should be necessary.**

*Water-Cement Ratio:*

Beyond the routine tests required by standard specifications the choice of cement is given little consideration, but in designing durable concrete the choice of the water-cement ratio is important. Concrete may be thought of as consisting of the aggregates surrounded and held together by a paste of cement and water.[1] It is not difficult then to understand that the durability of the concrete will depend to a very large measure on the quality and quantity of this paste. For given materials and consistency, the water-cement ratio will determine the quantity of cement and will, therefore, be a determining factor in the cost of the concrete. Hence, in deciding upon the water-cement ratio to be used in any work, the nature of the structure in respect to exposure the durability and structural strength desired, initial and maintenance costs should be considered. (See Figs. 10 and 11.)

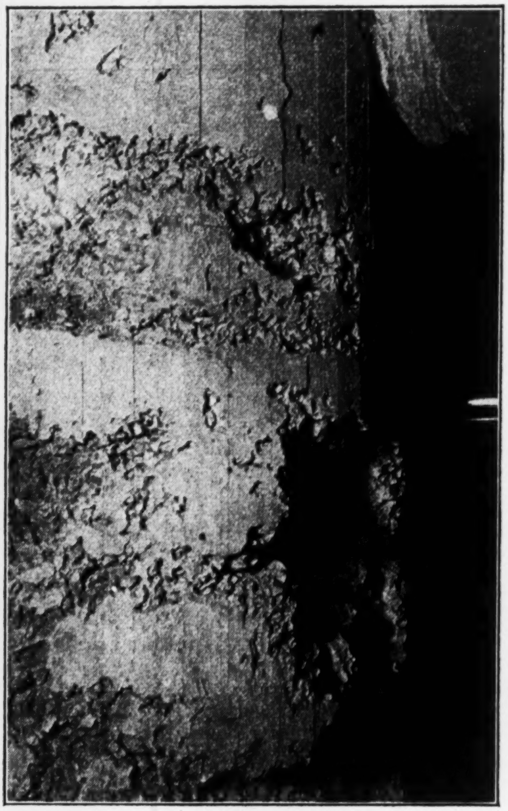

FIG. 12.—Showing Honeycombing.

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[1] F. R. McMillan, "Basic Principals of Concrete Making," McGraw-Hill Book Co., Inc., New York City (1929).

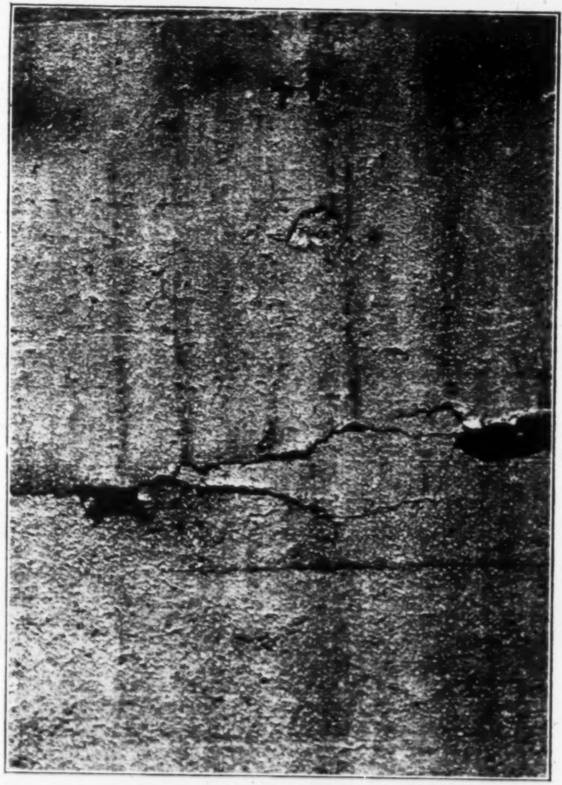

FIG. 13.—A Layer of Mortar Covering the Surface of Honeycomb Behind.

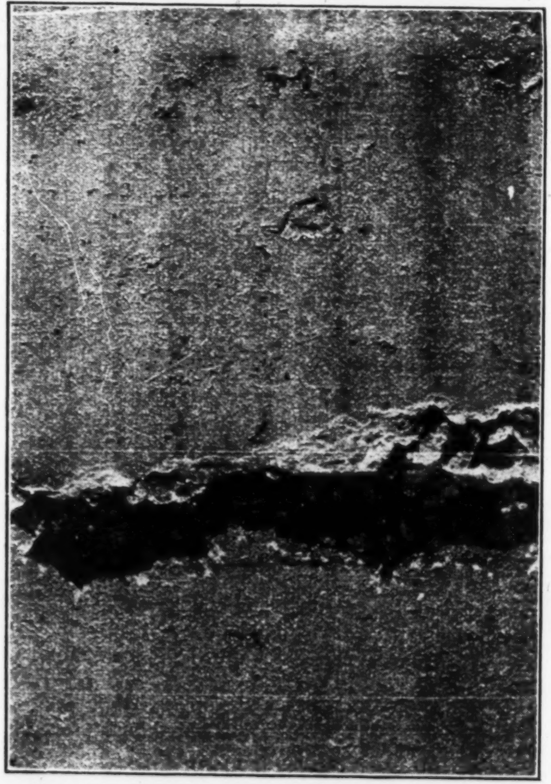

FIG. 14.—Same as Fig. 13 with Layer of Mortar Removed.

*Aggregates:*

If aggregates are sound and free from deleterious matter, except that which can be removed by washing, gravel or stone may be processed to give a concrete aggregate of suitable grading. Recent tests would indicate that the grading of the coarse aggregate within standard specification limits is not an important factor in determining the strength of concrete. The grading will, however, affect the workability of the mix and the most important consideration is to avoid an excess of material in the real mix between the $\frac{3}{8}$-in. and No. 8 sieves. If this is not done, unworkability in the form of harshness results which might be explained by saying that an excess of this material prevents the finer aggregate from coming into play.

In determining the maximum size of aggregate to be used on any piece of concrete work, due consideration must be given to the nature of the work, whether it be massive or composed of thin walls, beams, columns and floor slabs, carrying heavy reinforcement and where both are to be built provision must be made for the use of a small maximum size for the smaller members.

If the coarse aggregate is too large on reinforced work, segregation will result. The steel is usually close to the forms and if the clearance between bars, and also the clearance between bars and forms is not great enough, the aggregates may arch, preventing the complete filling of the form or resulting in only wet mortar passing. The result then is either a porous mass or a member covered with an envelope of mortar and laitance, which does not prove durable (Figs. 12, 13 and 14).

In the choice of maximum size of aggregate, further factors to be considered are the increase in cost of placing with increasing size of aggregate, the increase in cement necessary to maintain a given consistency with constant water-cement ratio as the maximum size is decreased, and the variation in cost of crushing and screening. If it were only more fully realized that a small saving in cement and crushing through the use of a larger sized aggregate caused increased placing costs, there is little doubt that the quality of much concrete placed would be greatly improved.

In preparing coarse aggregate, provision should be made to separate it into a sufficient number of sizes to insure a constant grading when mixing takes place. Where only one stock pile or storage bin is provided, considerable separation takes place which results in an uncontrollable and very troublesome variation in the workability of the resulting concrete.

In selecting a fine aggregate, the grading is important in determining the workability and there should not be an excess of material

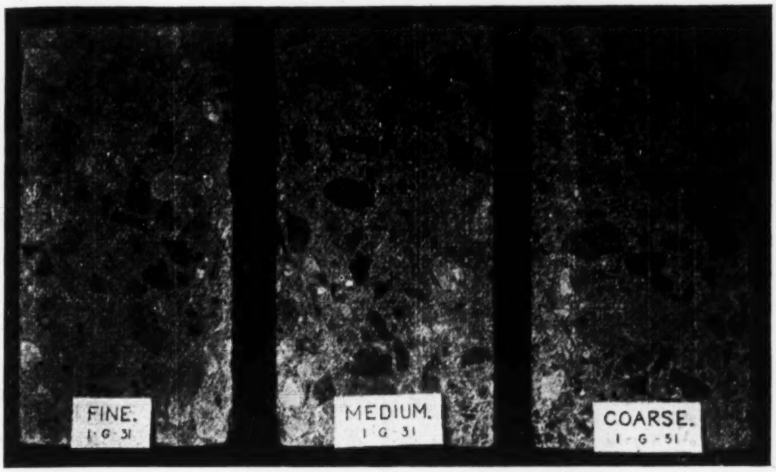

FIG. 15.—Three 6 by 12-in. Cylinders Which Have Been Sawed Lengthwise for Inspection.

The weight of all materials and the grading of the coarse aggregate was the same in all cases, the only variable being the grading of the fine aggregate.

SIEVE ANALYSIS OF AGGREGATES

| Fine Aggregate | | | | Coarse Aggregate | |
|---|---|---|---|---|---|
| Sieve | Percentage Passing | | | Sieve | Percentage Passing |
| | Coarse | Medium | Fine | | |
| ¼-in. Sieve | 100.0 | 100.0 | 100.0 | 2-in. Sieve | 100.0 |
| Sieve No. 4 | 90.2 | 96.5 | 100.0 | 1½-in. Sieve | 95.0 |
| Sieve No. 8 | 68.8 | 84.3 | 98.0 | 1-in. Sieve | 75.3 |
| Sieve No. 16 | 25.2 | 67.5 | 92.0 | ¾-in. Sieve | 52.5 |
| Sieve No. 30 | 16.4 | 41.0 | 73.0 | 1½-in. Sieve | 48.6 |
| Sieve No. 50 | 2.6 | 15.0 | 13.0 | ⅜-in. Sieve | 20.0 |
| Sieve No. 100 | 0.6 | 2.6 | 3.0 | ¼-in. Sieve | 5.0 |
| Fineness Modulus | 3.96 | 2.93 | 2.21 | .... | 7.32 |
| Slump, in | 2½ | 5½ | 1 | .... | .... |
| Flow | 126 | 156 | 115 | .... | .... |
| Weight per Cubic Foot, lb | 105 | 111 | 103 | .... | 104 |
| Weight per Cubic Foot of Dry Concrete, lb | 149.5 | 149.6 | 146.8 | .... | .... |

NOTE.—The slump and flow indicate the effect of the grading of the fine aggregate on the workability of the resulting mixes.

retained on any one sieve. A simple rule is that not more than about 60 per cent should be retained on any two successive sieves. To produce water-tight concrete there should be from 10 to 20 per cent passing the No. 50 sieve. The percentage of material coarser than the No. 8 sieve should be considered in relation to the grading of the coarse aggregate with a view to producing a workable mix (Figs. 15 and 16).

*Proportioning:*

In determining the proportions and quantity of fine and coarse aggregate to be used with a given quantity of cement paste, it should be borne in mind at all times that workability is absolutely essential if uniform and durable concrete is to be secured. The exact proportions will probably be best determined on the job and a satisfactory mix should readily be obtained if these are decided on by a competent person who realizes both the economies to be gained by reducing the percentage of fine aggregate and the increased labor and danger to good work, which will result from too small a percentage.

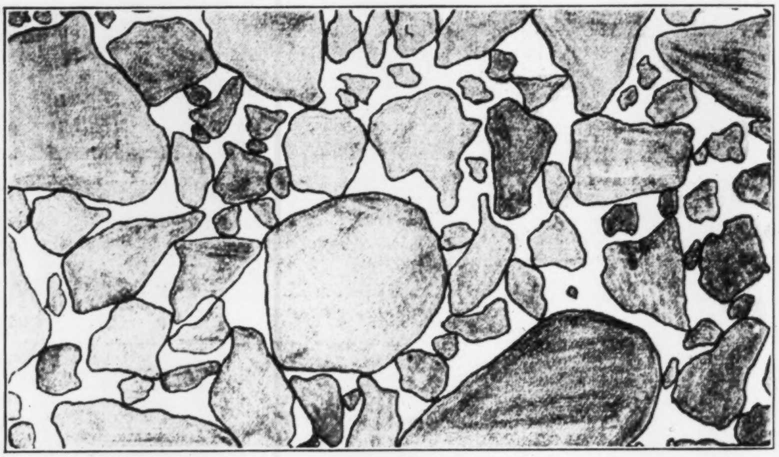

FIG. 16.—A Drawing in Which an Effort Has Been Made to Show How an Excess of Material of the Sizes Found Between the No. 8 and $\frac{3}{8}$-in. Sieves Produces Harshness and Prevents the Finer Material from Coming into Play.

*Mixing:*

In the matter of mixing, the time and speed are two important factors. The mixing must be carried out for a sufficient time to insure a homogeneous mass and from $1\frac{1}{2}$ to 2 minutes is considered a satisfactory time. Where longer mixing can be secured without interfering with other operations as is often the case, it should be encouraged. Where the mixing speed is variable, it should not be allowed to increase to such a point that there is any danger of the coarse aggregate being thrown to the outside of the mixing drum, and in that way causing a segregation in the mixer. This factor is important because some concretors have had the idea that by increasing the speed of the mixer they could reduce the time of mixing. Their concrete of course, would be anything but homogeneous.

*Transportation and Placing:*

When mixing is completed, the factors contributing to the final quality of the concrete are about half taken care of. From this point on the greatest care should be exercised to see that what has been done in the mixer is not undone in transporting and placing. Transporting and placing the concrete then consists of getting it to the point of final deposition in the same condition as when it came from the mixer. This is accomplished in many different ways depending on the nature of the work, and any method which does this without segregation, or too great a delay between mixer and forms should be considered satisfactory. Any work outside of compacting it in the forms is likely to cause segregation. A minimum of shovelling and spading should be attained through proper proportioning, mixing and transporting. Vibrators are coming into more general use, and where these devices are used to compact the material and to remove entrained air, their use is advantageous, but they cannot be substituted for the proper care in other operations. The capacity of the mixing plant and other facilities should be sufficiently great to allow a layer of con-

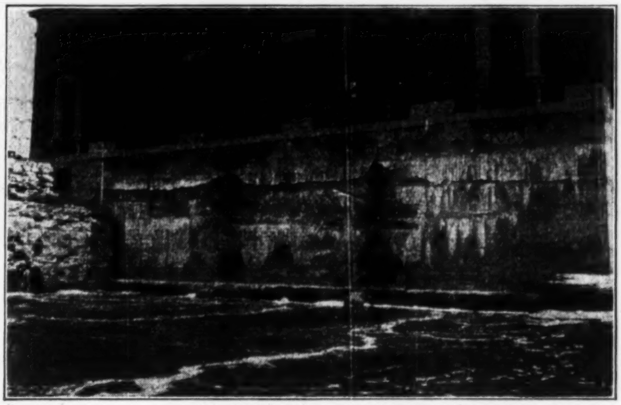

FIG. 17.—Showing Horizontal Joints Which Are Not Water-tight.

crete from 6 to 10 in. thick to be placed over the whole area often enough to prevent the formation of horizontal joints between lift planes. Many lift planes could be eliminated but where they are necessary every care should be taken to secure a good water-tight bond between successive lifts. (Compare Fig. 17 and Fig. 7.)

When the top of the form is reached in structures in which the top is to be exposed, it will be found to improve greatly the durability of the concrete if the form is over-filled, allowed to settle for a time, and then finished off with a minimum of trowelling.

After placing is completed, the concrete must be protected from drying and from freezing.

FIG. 18.—Top of a Breakwater in Which the Concrete Was Frozen Before Setting.

The nature of the work will again determine the exact requirements in the way of methods and equipment, but if the concrete is to gain its proper strength it must have water for hydration of the cement, and if the temperature is, say, 70° F., the mixing water will be evaporated before this is completed. Sprinkling or ponding with water, covering with a material which can be kept wet, painting or spraying with an asphaltic compound shortly after the initial set has been obtained to prevent evaporation are satisfactory methods of supplying or retaining the moisture required. In cold weather, heat must often be applied and this should be accomplished in a manner which will not dry out the mass. Unless precautions are taken, many methods of applying dry heat are unsatisfactory. A good method of curing concrete in cold weather consists of covering it with tarpaulins held a short distance from the forms and then applying steam through perforated pipes under the covers.

*Labor:*

Concreting in the past has been considered the work of the most unskilled labor, whereas most of the other divisions of building are carried out by men with training and experience. Contractors engaged in concrete construction would no doubt find it to their advantage if they made an effort to train their workmen and to find continual employment for these men. Through such a method, concreting would become an art or trade and the quality of the work would benefit accordingly.

## CONCLUSION

In this paper, an effort has been made to point out some of the weaknesses of concrete as made without scientific control and some of the steps to be taken to enhance the usefulness of this material; and, it may not be amiss to remark that in this world of ours nothing is permanent in the true sense of the word. The term is used by engineers only in a relative sense and the best of man's and nature's materials in time decay and disappear.

Year by year the use of concrete is increasing and it is interesting to note that it plays an important part in almost every structure, from the smallest to the largest, either being the main material used as in dams and pavements, or as a material necessary to the use of other materials, as in bridges and buildings. It is a relatively new material in many of its usages, and for that reason there are many things still to be learned concerning it. Structurally, it is one of the most flexible of building materials in the hands of the engineer for the modern-day requirements. It is too important a material in our present-day structures to do without, and it is not likely to be displaced by others for a long time, so if we are not satisfied with its performance, it behooves us to endeavor to improve it as well as our methods of using it.

There are a number of men scattered across the country today who have devoted much of their lives to a study of concrete and they are pretty well agreed that where the material is handled in a satisfactory manner, its performance comes up to expectations. What is possibly most urgent today is the necessity that the knowledge which has been gained by the few be spread to others where it can be applied more generally; and particularly must those in ultimate charge of concrete work be impressed with their responsibility for the application of the better methods.

From an inspection of a large number of concrete structures in service, one comes to the conclusion that of the total number of cubic yards of concrete placed, or of the total surface area exposed to the weather, only a small percentage undergoes deterioration rapidly; and this deterioration is seldom extensive over any one structure. The defective concrete is generally found in spots here and there in each structure showing that little blame can be attributed to the cement, or even the aggregates and as a rule a close inspection of the defective parts shows plainly that the methods of making and placing the concrete were at fault or that in exposed structures too small a quantity of cement was used.

*Acknowledgment:*

The author wishes to thank Mr. F. R. McMillan of the Portland Cement Association, Mr. R. B. Young of the Ontario Hydro-Electric Power Commission, and the Alcoa Power Co., for a number of illustrations which they so kindly supplied; and Mr. J. W. Lucas, a member of the author's staff, for compiling much of the material of this paper.

[For discussion of this paper, see page 835.—ED.]

# THE WEATHERING OF STRUCTURAL CLAY PRODUCTS: A REVIEW[1]

By J. W. McBurney[2]

## Synopsis

This paper reviews briefly the present state of our knowledge of the weathering of structural clay products. The mechanism of weathering as produced by temperature change in the presence and absence of water, wetting and drying, percolation of water, crystallization of soluble salts, and the action of the atmosphere are considered.

Among the tests for resistance to weathering considered are laboratory freezing and thawing, the sodium sulfate test and the autoclave test. Some attention is given to the possibility of correlating certain measured properties of structural clay products with the results of laboratory freezings and thawings.

## Introduction

Structural clay products, like all other materials, are affected, to a greater or lesser degree, in time, by climatic agencies. This paper endeavors to review briefly the present state of our knowledge of the weathering of structural clay products.

## Properties of Fired Clay Products

Before considering the action of weathering agents on structural clay products some attention should be given to the properties of the materials themselves. The raw clays, including shale, which are used to manufacture structural clay products become plastic on the addition of water and are formed into the desired shapes. On drying at ordinary temperatures, a material of slight strength results. Any rewetting will restore the original plasticity, and excess of water will produce mud. Plasticity is permanently lost (Brown(**1**)[3]) by heating to a temperature between 400 and 800° C. (750 and 1470° F.), the temperature required depending upon the raw material. It may be presumed that the point where plasticity is permanently lost also represents the point of maximum porosity as measured by water absorption (Knote(2)). As the temperature is increased, porosity and volume diminish (Bleininger(3)) and strength increases up to vitrification which is defined by Wilson(4) as "a partially fused condition of a fired piece of clay ware in which the pore space is small and the exterior volume is near a minimum." Orton(5) defines vitrification in terms of open pore space and sets a maximum of 3 per cent by weight for water absorption by 48-hour cold immersion or three-hour boiling.

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[1] Publication approved by the Director of the Bureau of Standards of the U. S. Department of Commerce.

[2] Research Associate of the Common Brick Manufacturers' Association of America at the U. S. Bureau of Standards, Washington, D. C.

[3] The boldface numbers in parentheses refer to the reports and papers given in the list of references appended to this paper, see p. 756.

The degree and nature of the porosity of structural clay products most certainly are related to their weathering resistance. Water absorption, while a measure of the degree of porosity, tells little of the nature of the pore structure. Penetrability and porosity measurements throw some light on the type of pore structure. Washburn(6) gives a classification of pore shapes as well as a definition and discussion of penetrability.

The picture which probably most closely illustrates the internal structure of a burned-clay body is that of an assemblage of closely packed spherical particles(7).

It would seem reasonable to assume that the method of forming the unit has an effect on the nature of the pore structure. McBurney(8) presents data showing much higher rates of absorption for dry-press bricks than for soft-mud bricks of the same total absorption. The water absorption of bricks measured by 5-hour boiling ranges[1] from less than 0.2 per cent to a maximum of 37.8 per cent.

The strength of structural clay products also covers a wide range. As with water absorption, strength is governed by the properties of the raw material and the degree of firing. Compressive strength,[1] determined on half bricks tested flatwise, ranges from a minimum of 800 lb. per sq. in. to a maximum of 28,000 lb. per sq. in.

The modulus of elasticity of bricks according to the Watertown Arsenal(9) ranges from 391,000 lb. per sq. in. to 4,820,000 lb. per sq. in.

In addition, the non-homogeneous condition of structural clay bodies very materially affects their behavior. Laminar structures, cracking, crazing, fissuring and nodules result from the use of certain raw materials with unsuitable methods of preparation and forming. Quantitative expression cannot be given to the relation between such departures from homogeneity and resistance to weathering, but unquestionably there is a relation. Kallauner(10) reports the effect of the presence of nodules on resistance to disintegration by freezing. Certain other properties of structural clay products are described in connection with specific types of weathering action.

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[1] **Unpublished data.**

## ACTION OF WEATHERING AGENTS

### *Temperature Change in the Absence of Moisture:*

The linear coefficient of thermal expansion of brick is given as 0.0000095 per deg. Cent. (0.00000527 per deg. Fahr.) by the National Physical Laboratory(11). The corresponding coefficients of thermal expansion for several mortars and concretes range from 0.0000072 to 0.0000126 per deg. Cent. (0.0000040 to 0.0000070 per deg. Fahr.) at room temperature according to Hatt(12) and Davis(13). The conclusion would seem warranted that no disruption or disintegration of masonry should be expected from differential thermal expansion of its components, assuming equal distribution of temperature. The rather low temperature gradients induced in structural clay products by climatic influences only would seem to offer a safeguard against disruption by differential expansion within a unit(14). The foregoing statements obviously do not apply to bodies such as architectural terra cotta and enameled brick if the glaze is poorly fitted, having a thermal expansion differing from that of the body or base. Crazing (effect of tension) or shivering (effect of compression) will result from temperature change. Differential volume change resulting from change in moisture content is, however, a more common cause of crazing than is thermal volume change. This is discussed in the next section.

There have been a few cases in which failure of burned-clay bodies took place by spalling where the circumstances and type of failure pointed to the fault being improper annealing. Internal stresses were undoubtedly present and evidently temperature change acted as the trigger to set off the failure.

Coulter(15) found that while moderately rapid cooling did not increase the rattler loss of paving brick, very rapid cooling caused undue loss by cracking. Norton's(16) discussion of the mechanism of spalling, though based on higher temperatures, applies also to the mechanism of disintegration caused by internal strain in general.

The phenomenon of "dunting" or "shivering" (Keeler(17)), though usually considered as originating in faulty composition, probably has differential thermal expansion as one of its factors.

### *Temperature Change in the Presence of Moisture:*

Cyclic wetting and drying and the effect of temperature on the crystallization of salts are discussed in other sections. Freezing and thawing is the action here considered. The weight of a cubic centimeter of ice is 0.917 g. while that of an equal volume of water is 1.000 g. Therefore, the freezing of water results in a 9 per cent increase in volume. When a porous body, more or less saturated with water, is frozen, it is essential that the water shall be permitted to expand during its conversion to ice. Since the pores are never completely filled with water, this expansion may take place to a more or less extent in the residual unfilled pore space. Obviously the smaller the residual pore space compared with the total pore space, the greater the tendency for disruption of the body when freezing takes place.

The ratio of "apparent porosity" (measured by water absorption) to "true porosity" was used by Hirschwald(18) and Kreuger(19) as indications of the amount of filled and unfilled pore spaces. "Schurecht's ratio"(McBurney(20)) uses the ratio of 48 hours cold absorption to 5 hours absorption by boiling. For brick, Palmer(21) proposes the ratio of water absorption, determined by such time of total immersion that the rate of gain in weight of the specimens was less than 1 g. during 24 hours, to the water absorption produced by 7 hours boiling.

The question of the usefulness of these ratios will be discussed under "Accelerated Tests for Weathering." For the present it is sufficient to remark that other factors undoubtedly must enter into consideration.

As noted by Orton(22), the presence of internal fissures and laminar structure strongly influences the behavior of structural clay products during freezing and thawing. Lack of relation between theory and observation can be explained by lack of homogeneity of the units examined. Certain burned-clay bodies have the property of extruding ice crystals from their interior through their surface. This may in part take care of the 9 per cent volume increase of water to ice.

There must also be considered the question of the probability of a body being saturated when exposed as part of a structure. Kreuger(23) reports that bricks removed from the face of a wall contain water equivalent to that absorbed during six days total immersion. Obviously, however, the probability of every freezing taking place while the units are saturated, or even wetted, is less than unity. There is no published experimental evidence bearing on the relative rates of disintegration as affected by variation in the degree of saturation.

*Cyclic Wetting and Drying:*

Numerous investigators have recognized that change in moisture content is associated with volume change for cements (Davis(13), White(24)), sand-lime brick, and certain building stones (Stradling(25)).

A distinction should be made between the behavior characteristic of cements, concretes, etc., where expansion on wetting and contraction on drying take place and these changes continue regardless of number of cycles or age of the specimen, and the behavior of bodies where expansion takes place during long-continued wetting and the original volume is not recovered except by heating at a relatively elevated temperature. Well-burned clay bodies are relatively free from the first effect (Stradling(25), McBain(26)). However, delayed crazing of glazed ceramic ware is sometimes caused by a gradual expansion of the body produced by the action of moisture. Semivitreous ware(27) with absorptions from 2.0 to 12.5 per cent showed expansions of from zero to 0.06 per cent on several months soaking. Whether the expansion produced by wetting is adsorption or rehydration remains to be determined (see discussion under "Autoclave Test").

It is certain that cyclic wetting and drying affects the durability of the bond between mortar and burned-clay masonry units (Palmer(28)).

*Percolation of Water:*

The effect on a well-burned, normally constituted burned-clay body of percolation of water is negligible. The primary effect on masonry is solvent action on mortar. The transportation and concentration of soluble salts may, however, produce injurious action on the burned-clay units. Certain clays, high in lime, can have their structure weakened by continued solvent action (Palmer(29)).

*Soluble Salts:*

In the absence of freezing, disintegration resembling frost action frequently takes place. The theory of this is as follows: The pores contain salts in solution which increases in concentration either by evaporation or addition. Finally, the solution becomes saturated and a lowering of temperature, or further evaporation, causes crystallization of the salts. The individual crystals and groups of crystals tend to enlarge, exerting pressure on the walls of the pores as in the case of ice crystals. English as well as Continental investigators have stressed the importance of crystallization of salt as a disintegrating agent of structural material. Relatively little attention is apparently paid to salt crystallization as a natural agency of disintegration in freezing climates. In the United States the sodium sulfate test seems to be considered as an accelerated test for freezing and thawing, rather than a simulation of an actual field condition (see section "Sodium Sulfate Test").

Disintegration may also occur when the materials in a body increase in volume as a result of chemical reaction. Air slaking of lime nodules represent a common cause of disintegration. The "sulfating" of lime and calcium carbonate is dealt with under another section.

*The Atmosphere as a Disintegrating Agent:*

For the United States, the abrading action of wind-blown sand may be neglected during the average life of a building. The most important effect of the atmosphere is the "sulfating" of lime compounds by the action of sulfur dioxide. Sulfur dioxide can be considered a "normal" constituent of the air of the cities (30). Searles-Wood calculates that 75 per cent of the sulfur in coal escapes into the air. He calculates that 16,000,000 tons of coal would convert 500,000 tons of limestone into 860,000 tons of gypsum. The process is as follows: Clay products may contain lime originally present in the clay or acquired from the mortar. The carbon dioxide content of the air quickly carbonates this lime, forming insoluble calcium carbonate. The sulfur dioxide in the air acts upon the calcium carbonate converting it to calcium sulfate which is somewhat soluble and may dissolve and recrystallize, exerting crystal pressure as described in a previous section.

Well-burned clay bodies, according to McBain (26), are practically free from the tendency, shown by certain other materials, to gain and lose moisture with variation in atmospheric humidity. The only objectionable effect produced on clay products by high humidity is that of retarding the evaporation of water, and producing condensation when the dew point is passed.

*Biological Action:*

Well-burned clay products appear to be unaffected by biological agencies. Cases have been reported where certain types of bacteria have produced sulfating of mortars with consequent disintegration by crystallization action on the bricks (Rawn (31)).

## ACCELERATED TESTS FOR WEATHERING

*Freezing and Thawing:*

There is at present no recognized standard for conducting freezing-and-thawing tests on structural clay products aside from the requirements for the test appearing in the Society's Standard Specifications for Drain Tile (C 4 – 24) (32). The methods of test by freezing and thawing given in the specifications for load-bearing structural tile (33) are hardly definite enough to be considered a standard. Jones (34) recommends that the choice of freezing while totally immersed, partially immersed, or entirely in air, should depend upon the expected location of the unit in the structure.

The method used by the author is as follows: The test specimens are half bricks from the transverse test. The water absorption of these specimens is determined after 48 hours' cold immersion and also after 5-hour boiling subsequent to the 48 hours of immersion. After drying the specimens for 48 hours in a gas-fired oven at 110 to 140° C. (230 to 284° F.), they are totally immersed in tap water at room temperature for 48 hours. They are then set on edge in ½ in. of water contained in shallow pans and placed in a freezing chamber at −15 to −6.7° C. (5 to 20° F.) for 20 hours. The bricks are then thawed by 4 hours' total immersion in tap water at room temperature and the freezing and thawing repeated until three cycles have been completed. The specimens are again dried for 48 hours in the oven, again soaked for 48 hours and three more freezings and thawings given. This alternation of three freezings and thawings and one drying is continued until, (1) the half brick separates; (2) a total loss of 10 per cent of the dry weight takes place, or (3) three hundred cycles have been completed. The specimens are weighed to the nearest gram every drying, and a log is kept with considerable detail describing the progressive changes in appearance of the specimen.

It would appear that both research and standardization are needed in respect to the freezing-and-thawing test. Some of the factors to be considered as possibly influencing the rate of disintegration by freezing and thawing are: the degree of saturation of the test specimen, the degree of immersion during freezing, the rate of freezing, the method of thawing and the effect of repeated drying.

*Sodium Sulfate Test:*

Brard(35) proposed the use of sodium sulfate to simulate the disintegrating action of weathering agents. Reference is made to Orton's(22,36) reports for a description of the sodium sulfate test. Orton's technique has, however, been considerably modified by other investigators.

Staley(37) stated in 1918: "Further there are many *possible variations in the method of making the test, and it has never yet been standardized*" [italics his]. The duration of immersion of the test pieces in the solution, the temperature of the solution during immersion, the temperature and duration of drying and the percentage of sodium sulfate in the solution are among the unstandardized factors listed by him.

Disintegration caused by crystallization of salts is unquestionably a very important factor in natural weathering (Furlong(38), Jack(39)). However, the sodium sulfate test has seemed to interest investigators in the United States not as a simulation of natural disintegration by salt action but rather as an accelerated measure of resistance to freezing and thawing. They attempt to express the results of the sodium sulfate test in terms of freezing-and-thawing tests. For example, Orton(22) considers that "The data at hand, therefore, seem to warrant the statement that four sodium sulfate treatments will, in the vast majority of cases, subject a brick to a test as rigorous as twenty-five actual freezings under the conditions described above." Staley(40) reports: "One treatment by this method (his sodium sulfate test) is approximately equal to two treatments by the partial-immersion freezing method." The same investigator states in Schlick's(37) report: "Between the limits seven to eleven per cent absorption, one sulfate treatment approximately equaled three standard artificial freezing-and-thawing treatments."

However, the same investigator states on page 62 of Schlick's report: "The sulfate treatments showed greater effects when compared with freezing-and-thawing treatments, as the tile became more dense and resistant for the reason that the test is more severe. Therefore, there can be no exact parallelism between sodium sulfate and freezing-and-thawing tests over a wide range of absorption." In other words the relation between sodium sulfate tests and freezing-and-thawing tests is not linear.

The Building Research Board(41) does not consider that the sodium sulfate test measures freezing and thawing resistance, but uses the test in the study of crystallization attack. Magnesium sulfate and sodium thiosulfate solutions are also used. The report states that the action of these salts is less severe than that of the sodium sulfate.

### *The Autoclave Test:*

Schurecht(42) proposed the testing of glazed ceramic bodies by autoclaving at a steam pressure of 150 to 175 lb. per sq. in. for one hour. He considers that such a treatment will produce immediate crazing of the glaze if the ware tested is of such a nature that crazing would appear naturally after some years exposure. The same investigator(43) measured the linear expansion produced on certain ceramic bodies by soaking in cold water up to 312 hours, boiling for 4 hours and autoclaving for one hour each at several steam pressures. The changes in length caused by drying at 110° C. (230° F.) after each of these treatments were also measured. He found for one of the clay bodies that no expansion whatsoever was produced by these treatments. The other clay body (open-firing) expanded up to 0.008 per cent after 312 hours soaking and reached a maximum of 0.03 per cent increase after autoclaving at 125 lb. per sq. in. steam pressure. Additions of silica to the clay body materially increased the expansion.

Drying, after soaking, increased this expansion in most cases when compared with the wet specimen.

One conclusion is: "Increasing the firing temperature of ceramic bodies reduces their moisture expansion to a marked extent in most cases."

Mills(44) investigated the effect of varying pressure, time and position of the test specimen. He notes the cumulative effect of repeated tests in producing crazing.

Everhart(45) compares the results of the autoclave test with the freezing-and-thawing test and the sodium sulfate test on glazed brick. He points out that crazing is the result of the autoclave test, while spalling is produced by the other two tests.

There appears to be some uncertainty as to the mechanism of these volume changes. Washburn(46) considers that prolonged contact with water at elevated temperatures brings about partial rehydration of a fired clay. This would presumably account for the expansions observed by the several investigators. On the other hand, Holscher(47) found expansion to result from boiling in carbon tetrachloride, paraffin, xylene, kerosine and petrolatum. He comments: "The data point to the probability that the increase in length may be due to causes other than the rehydration of any constituent in the body." Volume change due to gel adsorption of the type considered by McBain(26) is probably the explanation of the behavior of underburned clay bodies.

The autoclave test is apparently at present used only as an accelerated test for crazing of glazed ceramic wares. But there exists the possibility that measurements of the volume changes produced by autoclaving or volume changes caused by a few freezings and thawings may serve as measures of the ultimate resistance to freezing and thawing. The Building Research Board(48) has made some attempt to correlate volume changes during freezing and elasticity with resistance to weathering. Their investigations were for the most part on building stone.

## PREDICTION OF WEATHERING RESISTANCE FROM LABORATORY TESTS

It should be evident from the preceding sections that weathering is not a simple uniform process. The development of a laboratory method for estimating weathering resistance possibly should await the standardization of climate and exposure. However, the real situation does not warrant such a degree of pessimism. Many generations of observation on the part of the manufacturers of structural clay products have produced the generalization that a well-burned clay body has satisfactory weathering resistance while trouble is usually associated with the ware from the cooler portion of the kiln. The simplified problem thus becomes: How can under-burned ware be readily identified? But to define well-burned and underburned in terms of resistance to weathering begs the question. What are the measureable properties of burned clay bodies which vary with variation in weathering resistance?

For products of a specific clay or shale molded by the same process into units of the same size and shape there are a number of measureable properties, variation in which parallels variation in degree of burning and degree of resistance to weathering.

Strength, porosity, density and color do not at all exhaust the list. But if variation in any one of these properties were to be plotted against some scale of weathering resistance it would be found that when different clays and shales were considered, each clay and shale might have its own characteristic curve.

TABLE I.—COMPARATIVE DATA ON SHALE AND SURFACE-CLAY BRICK.

A, Mid-western shale brick.
B, Eastern surface-clay brick.

| Manufacturers' Grade of Brick | Flat Compressive Strength, lb. per sq. in. | | Modulus of Rupture, lb. per sq. in. | | Water Absorption, 5-hr. Boil, per cent | | Ratio of 48-hr. Cold Absorption to 5-hr. Boil Absorption | |
|---|---|---|---|---|---|---|---|---|
| | A | B | A | B | A | B | A | B |
| Dark | 16 530 | 4280 | 1660 | 545 | 3.78 | 16.34 | 0.57 | 0.62 |
| Medium | 12 540 | 3915 | 1385 | 490 | 9.06 | 16.70 | 0.81 | 0.63 |
| Light | 7 180 | 2175 | 925 | 350 | 13.98 | 20.38 | 0.86 | 0.68 |
| Salmon | 4 500 | 1360 | 590 | 245 | 17.60 | 21.24 | 0.91 | 0.70 |

Table I compares a certain midwestern shale "A" with an eastern surface clay "B." Without reference to Weather Bureau records, it can be said that the two plants serve territories of about equal severity of weather.

Neither manufacturer recommends his salmon grade for exterior exposure. Both manufacturers guarantee their first three grades. The very interesting fact is that 42 cycles of freezing and thawing have caused a loss of weight on the "B" salmons of about 0.5 per cent of their dry weight while all of the "A" salmons have been destroyed, the average endurance being six cycles for the first four of the five specimens tested.

Comparing the grades of manufacturer A with manufacturer B it is evident that strength and water absorption provide no measure whatsoever for judging the relative resistance to freezing and thawing of these two salmon grades. The only clue to their behavior is provided by their ratios of 48-hour cold water absorption to 5-hour boil water absorption. Obviously no generalization can be established on but two samples. But from the scattering and incomplete data at present available, the conclusion is tentatively offered that a brick with a 48-hour cold absorption to 5-hour boil water absorption ratio of 0.90 or over has rather little chance of surviving 50 cycles of freezing and thawing conducted according to the method described earlier in this paper.[1] On the other hand a ratio of 0.80 or under so far has not been associated with a failure. These conclusions are based upon from nine to fifty-four cycles given to 450 specimens representing about 80 plants. About 400 of the 450 specimens are regarded as "salmons" by their makers. It is evident that structure (presence or absence of nodules, lamination, fissures, etc.) has an important part in the behavior of these specimens. The performance of the specimens has ranged from complete destruction by one freezing to a loss in weight of less than one part in twelve hundred after 18 dryings and 54 freezings and thawings. A total of 60 specimens has so far failed.

## Conclusions

Experience has shown that a "well-burned" structural clay body usually has high resistance to weathering and such ware can usually be separated from underburned ware by visual inspection when a single raw material is considered and the entire range of the kiln is known. But the properties and characteristics of structural clay products are not yet sufficiently related to weathering resistance to enable an opinion to be rendered concerning a specific material where its history is unknown and the only data available are its strength and water absorption. Laboratory freezing-and-thawing tests are usually out of the question as a routine examination on account of the time required. But where an unknown material is in consideration, proven ability to withstand at least 50 cycles of freezing and thawing should be required for reasonable assurance.

In the way of recommendations, the following is offered:

There should be much more study of weathering in the field. What was the weathering agent? What was the condition of exposure? What was the result? The freezing-and-thawing test should be studied as a preliminary to standardization. By field observation it should be possible to evaluate this standardized freezing-and-thawing test in terms of time of exposure to climates of various degrees of severity. Having a "calibrated" freezing-and-thawing test, the utility of various other accelerated tests and indirect measures can be judged. It is the author's conviction that such measures and tests exist.

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# NOTES ON THE WEATHERING OF NATURAL BUILDING STONES

By G. F. Loughlin[1]

## General Statement

In this paper it is intended to focus discussion on certain factors of critical importance rather than to present a thorough outline of the subject. The majority of these factors are of critical importance only as affecting superficial details rather than as affecting the ultimate durability of stone in structures of different kinds, but superficial details are likely to attract the most attention and therefore to assume great importance in the selection of stone.

The weathering of stone depends primarily upon its mineral composition, texture, and structure, and secondarily on the conditions under which it is used. Some minerals are very stable under all ordinary conditions of use, but others are very unstable and a small percentage of them in a stone may be sufficient to affect seriously both superficial detail and even ultimate durability; again a stone may consist entirely of stable minerals but its cohesive strength may have been originally low or it may have been seriously decreased by microscopic fracturing of natural or artificial origin.

## Granites and Related Igneous Rocks

### *Original Minerals:*

The minerals that originally constitute the common igneous rocks, for example, granites, and diorites, gabbros, and diabases, commercially known as "black granites" and as "trap rock" in part, are nearly all quite durable from a practical viewpoint, if free from alteration when the stone is quarried. Quartz, feldspars, hornblendes, pyroxenes, and even most of the micas can be dismissed from further consideration. Inferior granites owe their inferiority not to these minerals but to secondary minerals derived from them through deep-seated processes of alteration, or through age-long shallow processes above the level of ground water. These shallow processes are especially effective on stone that has already undergone deep-seated alteration.

The original micas of granites, though frequently mentioned as sources of weakness, may be seen in granites that have been exposed for decades without any noteworthy appearance of weathering. This statement is especially true where the mica flakes are small and evenly distributed. Larger, tablet-like crystals $\frac{1}{8}$ in. or more in diameter in certain granites of inferior quality have been known to swell during prolonged exposure and finally to "pop" out, leaving a pitted surface. This process is not uniform in different granites of similar texture, and the greater tendency of the mica crystals to swell in some granites than in others may be due to a natural internal strain that has opened cleavage planes of the micas or to the bruising of the micas during the finishing of the granite. Micas in coarse-grained, gneissoid granites may be so concentrated in streaks or lenticular bunches that they also become sources of weakness under similar natural or artificial conditions.

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Distinction between white and black mica has been deliberately avoided thus far, as their physical properties, which are very similar, are far more important than their chemical properties. Well crystallized white mica is chemically very stable, and black mica, which contains ferrous iron, is more stable under ordinary conditions of weathering than some have supposed it to be. The author does not recall any case in which oxidation of iron in a previously unaltered black mica can be reliably attributed to the attack of air and rain after the stone containing the mica had been placed in the wall of a structure. Concentrated attack by acid or sulfate waters that percolate through the floors and the backings of walls to the outer surface of the stone may appreciably oxidize the iron and produce a rusty stain that will be limited on the soundest, impervious granites to the extent of capillary spread from masonry joints; but this stain may appear elsewhere on granites that have been rendered relatively permeable through deep-seated chemical alteration or mechanical deformation. Some varieties of black mica are more subject to discoloration than others. Hornblendes and pyroxenes also are subject, theoretically at least, to discoloration under similar circumstances, but to a less degree, and no noteworthy occurrence of such discoloration with which the author is acquainted can be attributed to these minerals.

### *Significance of Minerals Formed by Deep-Seated Alteration:*

The most common minerals formed during the deep-seated alteration of granites and other igneous rocks are microscopic, white to pale green mica (sericite), green micaceous chlorite, green epidote, and calcite and related carbonates. Pyrite, the iron disulfide, may accompany these minerals. These minerals commonly occur together and replace the original feldspars and black minerals of the rock to some degree. They have been formed in some places, particularly in the vicinity of ore deposits, by the action of solutions that mark the closing stages of igneous or volcanic activity, so that the igneous rock is altered almost as soon as it is formed. Similar solutions, regardless of their exact origin, may attack rocks that had been free from alteration for ages. The degree of alteration depends upon the quantity of solution and the number and distribution of fractures that permit permeation of the rock. Very few granites, or other igneous rocks, are entirely free from this kind of alteration, but in granites of recognized merit as building and monumental stone, conspicuous effects of alteration are confined to narrow streaks along the margins of tight fractures and to a minor amount of microscopic impregnation of the feldspars and black minerals throughout large masses of the rock. What appears under the microscope like a considerable development of this kind of alteration may produce no conspicuous effects after prolonged exposure, and occasional corrosion tests by the author have produced no noteworthy results; but a sufficient development of such soft and flaky minerals as sericite and chlorite and of so soluble a mineral as calcite will reduce the granite's ability to take and maintain a good polish, and may even result in a pitting of the surface.

Probably the most serious effect, however, is to render the granite more permeable. The permeability may still be slight compared with such highly porous stones as our best known limestones and sandstones that are used for building, but it affords more access to acid water that percolates above ground-water level as well as water that works through from the back to the outer surface of a building wall.

### *Significance of Minerals Formed by Weathering:*

The most significant minerals produced by the weathering of granitic rocks prior to quarrying are those of the clay group, also the brown, yellow, and red hydrated oxides of iron. The iron oxides are confined mainly to fracture surfaces where oxygen is available to oxidize iron as soon as it is dissolved from any iron-bearing mineral. They produce the "seam-face" surfaces and are themselves very durable minerals. The durability of the "seam-faced" stone depends upon the durability of the other constituent minerals. Where the iron oxides have penetrated for an inch or two into the stone it has partly or completely filled minute cracks and thereby tended to toughen the stone and decrease its permeability. Paving blocks have been rejected at times because of these brown "sap" stains, but actual tests may prove that the stained stone is quite as resistant to abrasion as the unstained stone, and may even be superior.

Minerals of the clay group have been generally regarded with suspicion. Recent studies of this group, however, have shown that it is much more complex than formerly realized, and that certain clay minerals are harmless from the standpoint of weathering, whereas others, notably of the leverrierite group, are among the most unstable of all minerals because of their capacity to absorb moisture and swell to a remarkable degree. This group of minerals, including montmorillonite, beidellite, and nontronite (an iron-clay), are formed by the attack of acid and sulfate waters on silicate minerals that contain aluminum, and the finely flaky or fibrous sericite and chlorite in altered granites are especially subject to attack. The resulting clay minerals have so strong a resemblance to sericite, however, that their presence may readily be overlooked, even during microscopic study; but they can be distinguished by a competent petrographer.

A small amount of these clay minerals supplements such minerals as sericite, chlorite, and calcite in softening a granite and rendering it subject to weathering by natural or artificial agencies. The remark has been made that polishing "burns a granite up," but that some granites nevertheless take a permanent polish. Those granites that take a permanent polish are, according to the author's experience, free from noteworthy alteration, either due to deep-seated changes or to weathering; but certain other granites that have not maintained a good polish have undergone both kinds of alteration to some degree.

Some at least of the granites that tend to "blister" or thinly scale along finely bush-hammered surfaces have been softened by the effects of deep-seated alteration. It is conceivable that others, although escaping marked chemical alteration, have been so subjected to compression by natural forces that their rift and grain are unusually well developed and cause too ready a yielding to the force of hammering as well as to the spread of stains either from mortar joints or from other external sources.

Discussion could be extended to cover other alteration products in igneous rocks, such as the zeolite group of minerals, but these are comparatively so unusual that no time will be taken for them here. The effects of weathered rock on excavating and tunnelling is a closely related topic of interest to engineers, but this also is dismissed with the remark that difficulties in excavation through granitic and other igneous rock as well as gneisses and schists are likely to be directly related to zones of excessive fracturing and alteration.

## MARBLES AND LIMESTONES

### *Comparative Significance of Dolomite and Calcite:*

Marbles and limestones, since they consist essentially of carbonate minerals, are considered together. Most of the marbles and practically all of the limestones used for buildings and monuments today are of the calcite or high-calcium variety. The dolomite or high-magnesium varieties that are sufficiently attractive for these uses and sufficiently adapted for economical quarrying and finishing are comparatively few, although the mineral dolomite is harder than calcite and commonly regarded as more resistant to corrosion. Dolomite, however, commonly contains a small amount of iron carbonate that is subject to oxidation and discoloration, especially on surfaces that have been finely tooled or cleaned by sandblasting, and even though it does dissolve more slowly than calcite it may show the superficial effects of weathering much sooner. Rocks composed mainly of dolomite are more brittle and may be too much fractured to permit quarrying, although calcite marble or limestone close by may be well adapted for quarrying. Dolomite marbles unaffected by these chemical and structural difficulties resist corrosion much longer than calcite marbles exposed under similar conditions, and should be especially suited for exterior use.

If a marble composed of granular dolomite is attacked by acid waters, either in the ground or in the laboratory, corrosion takes place slowly along grain boundaries, gradually loosening them and reducing the marble to a dolomite sand. A calcite marble subjected to the same treatment is more likely to dissolve so rapidly and completely that no loose grains can accumulate. A cursory glance at the end products might give the impression that the dolomite marble had broken down whereas the calcite marble had resisted attack. Obviously, the original bulk and outline of each sample of marble must be known before a true comparison of resistance to corrosion can be made. It may be that such cursory glances have prompted the remark that a marble or limestone, because it contains 97 or 98 per cent of calcium carbonate (calcite) is highly resistant to chemical attack, when as a matter of fact calcite is the most soluble of the common rock-forming minerals except gypsum.

The difference in rate of corrosion between calcite and dolomite is well shown in certain dolomite marbles that contain crystals of lime-magnesia silicates, notably tremolite and diopside. The formation of these silicates involved the removal of more magnesia than lime from the dolomite, leaving the remainder of lime to recrystallize as borders of calcite around the silicate crystals. These borders of calcite dissolve more rapidly than the dolomite, and aid in the loosening or "popping out" of the silicate crystal. The silicates themselves have splintery structures that are somewhat weakened during the finishing of the stone and thereby increase the tendency to "pop."

"Sand spots" that are present in some dolomite marble are the result of slow corrosion, and in some places at least are due to attack by sulfuric acid that has been generated by the local decomposition of pyrite grains before the stone was quarried. If these pyrite grains are partly decomposed before the stone is quarried, the "sand spots" are likely to appear during the sawing or finishing of the marble and the process of decomposition may continue so long as moisture in the marble is available to attack the pyrite. Whether or not the process will be continued appreciably by rain water that lodges in the corroded or "sandy" spots is doubtful, and depends in part upon the size of the pyrite grain. Microscopic grains are attacked with comparative readiness, whereas grains $\frac{1}{16}$ in. in diameter appear to have resisted corrosion for more than 50 and even as much as 100 years. Decomposing pyrite grains in a calcite marble are more likely to produce small empty cavities rather than "sandy" spots.

*Rate of Corrosion of Calcite Marble and Limestone:*

Calcite marble, after exposure to rains for 20 years or more, commonly shows a noticeable roughening of smooth surfaces, which gradually develops a loosening of grains, a process similar to that described under dolomite but with very much weaker solvents. The degree of crumbling depends upon the size of grains and the permeability along their boundaries. Fine, even-grained marbles are likely to show crumbling before those of coarser, uneven, and interlocking grain. Statements have been made that certain calcite marble tombstones have their inscriptions well preserved after 100 years, and that the marble therefore has resisted corrosion. The author has seen tombstones that have been markedly corroded over parts of their surfaces and untouched in others, and the explanation is that the untouched places have been protected from direct attack by rain. If inscriptions are protected by overhanging parts of the monument, or if tombstones in old graveyards are protected from the rain by mats of tall grass they will be preserved indefinitely.

As the principal limestones quarried for building also consist of calcite, they are corroded as rapidly as marble, but owing to their lack of "sugary," crystalline texture no crumbling is likely to accompany corrosion. Marble and limestone have been seen after exposure side by side for more than 50 years, the marble, which was partly protected, showing a little crumbling and the limestone none; but the limestone on close inspection showed appreciable corrosion, especially on its more exposed surfaces.

Experiments on corrosion of limestone under different conditions show that a stone fully exposed to a rainfall of 40 in. a year and an atmosphere uncontaminated by excessive smoke would have its surface dissolved to a depth of about 1 cm. or 0.4 in. in 450 years. Conditions in the average large city increase this rate. Observations in New York City indicate a rate of about $\frac{1}{16}$ in. in 50 years, the effects of corrosion being distinct on limestone of uneven grain but only noticeable on close inspection on limestone of fine, uniform grain. This rate is negligible when the lives of most buildings are considered, but is very important with regard to the preservation of details of finish and ornamentation. It may be repeated that where limestone and marble are not exposed to rainfall, or only to occasional rains from unusual directions, no corrosion, or only slight corrosion, is developed even through long periods of time, no matter how humid the climate.

*Selective Use of Marble, Limestone, and Granite to Counteract Corrosion:*

Special conditions of attack by more concentrated acid or other solutions require special consideration, but in general it may be suggested that under such conditions a superior granite or siliceous sandstone be used instead of marble or limestone. One condition sufficiently widespread to deserve comment is the use of a marble or limestone pedestal beneath a bronze statue, or of either stone in contact with bronze or copper fixtures of any kind. Every rain dissolves an appreciable quantity of copper from these statues or fixtures. The dissolved copper on reaching the marble or limestone promptly reacts with it and is deposited as the green copper carbonate which replaces an equivalent amount of the stone. No such reaction takes place between the copper solution and the minerals of a good granite or impervious, siliceous sandstone. It may be suggested, therefore, that a marble or limestone pedestal be capped with an overhanging slab of granite that will protect it from the drip of copper solution.

Similarly seepage from the ground tends to corrode marble and especially porous limestone within the range of capillary rise. The advisable practice is therefore commonly followed of laying a base course of granite. Seepage may develop blistering in some granites to the slight depth of the cracks formed by the finishing tools, but thereafter corrosion on granite is negligible. Discoloration is likely to take place on any stone affected by seepage from the ground as well as from mortar joints and backing, but the amount of corrosion associated with it is much less on a good granite than on any marble or limestone.

## SANDSTONES

The durability of sandstone depends largely upon the material that cements its grains together. Of the four common cements, silica and pure iron oxide are ordinarily durable, but calcite and clay, whether or not accompanied by silica and iron oxide, are subject to rapid weathering. The weathering of calcite cement hastens the removal of superficial details, but, as in limestone, the wearing away of the surface as a whole is too slow to affect the strength of the stone appreciably. The conspicuous effects caused by the falling away of sand grains left unsupported by the removal of calcite cement, however, are in marked contrast to the almost invisible effects of weathering on a fine, even-grained limestone that consists entirely of calcite.

The same clay minerals mentioned in the discussion of extremely weathered granite constitute the principal cements of some sandstones and form considerable percentages of some impure limestones. Obviously these stones are likely to weather rapidly, although the rate depends largely upon the distribution of the clay and its accessibility to rain and air. Where clay, whether soft or hardened into shale, is concentrated in films or thin layers along bedding planes, the stone is locally weakened and likely to admit water that is absorbed by the finely porous clay, which tends to slake or, in frosty weather, to expand and cause the stone to split. Clay evenly distributed throughout the stone tends to absorb moisture, but the force of its expansion due to slaking is negligible except close to the surface. In freezing weather, however, it may cause the stone to burst. A large percentage of evenly distributed clay cement is likely to promote a gradual though rather rapid wearing away of the surface, but in some stone the clay is so thoroughly inclosed in siliceous or carbonate cement that its absorptive and therefore slaking powers are negligible.

Although some of the most extreme cases of weathering on building stone have been due to clay minerals, judgment on a stone containing clay should not be hastily passed without determining the kind of clay present. Some clay minerals are remarkably durable, and even the others may have been made durable by natural processes of baking or dehydration, especially near contacts with igneous rocks. A fair appraisal of a sandstone's weathering qualities may require even more thorough study than that of any other stone.

## GENERAL CONCLUDING REMARKS

The author, in his effort to present certain outstanding features of weathering, has thus far given no more than incidental consideration to such well-known properties as hardness, porosity, resistance to changes in temperature, and permanency of color. These properties are generally recognized by those interested in stone, and need not be reviewed at length, but the need of sound judgment in the evaluation of these properties cannot be too strongly emphasized. For example, a high percentage of porosity in a stone is not necessarily a detriment so long as the stone has been thoroughly seasoned and is not used in places where it will be frequently or continuously saturated with water; again, equal degrees of porosity in stone containing undesirable clay minerals and those composed largely of calcite or of quartz and durable silicate minerals should be evaluated very differently.

Porous stone has been used even in the submerged parts of bridge piers without noticeable deterioration, although one cannot help questioning the advisability of such a use. A coarsely porous stone of adequate strength, not subject to injury by frost, and composed of insoluble minerals may prove quite satisfactory. Limestone also may prove satisfactory if the river water flowing by the piers is already saturated with lime, as is the case throughout much of the Mississippi Valley. Along the eastern seaboard, however, river waters have a marked capacity for dissolving carbonate of lime, and the long-continued soaking of a porous limestone is likely to reduce the strength of the stone appreciably. Just how serious these conditions may become, obviously depends on the load supported, the effect of fatigue, and perhaps other factors. The author knows of no failures of bridges due to the crushing or collapse of immersed stone, but the question is worthy of thought.

In a discussion such as this where attention is focused upon decomposition and disintegration, there is danger of giving too pessimistic an impression, and it should therefore be emphasized again that most of the processes that have been discussed affect only the appearance of stone and are too slow to affect its strength seriously within the "lives" of most buildings. The weathering properties of stone, furthermore, constitute only one of many groups of factors that must be dealt with in utilization. For some uses weathering properties may be paramount, and for others negligible; again immediate and ultimate costs as well as the object in view may outweigh differences in weathering properties, especially as the majority of noticeable weathering effects are superficial. Some stones are vastly superior to others for certain uses, but all may compete on equal terms for other uses. It is, therefore, to the ultimate benefit of all, producers and consumers alike, that different kinds of stone are selected with due regard to their inherent qualities and the relative importance of these qualities for the uses in mind. New questions regarding weathering under certain conditions arise every now and then and should be promptly investigated in order that sound judgment in the utilization of stone may keep pace with general progress.

# THE WEATHERING OF SLATE

By C. H. Behre, Jr.[1]

As slate is, strictly speaking, a rock, a discussion of its weathering properties separately from those of rocks in general is justified only by a consideration of those qualities in which it differs strikingly from most other commercial stone. In what follows three objects are especially in mind. First, in view of the fact that its properties vary with different directions in the slate block, it is desired to show how this directional difference must be recognized in weathering tests of slate. Second, attention is called to the convenience of the polarizing microscope as an adjunct in testing. Third, it is desired to suggest lines for further research that may bring out weathering qualities in one slate in comparison with another.

It is well first to point out one marked difference between slate and other building stones. In most rocks used in construction, physical or chemical alterations show no directional variations or slight ones at most. In slate, however, many properties exhibit noteworthy quantitative differences, depending on whether they are measured parallel or at right angles to the cleavage planes. This peculiarity is occasioned by the arrangement of the minerals making up the rock. Slate is composed chiefly of lath-shaped, lenticular, or leaf-like crystal grains—generally the minerals mica, chlorite, and quartz. These possess "dimensional parallelism"—the least dimension is transverse and the other two or the greatest are parallel to the cleavage planes. It is this that gives to the rock that singular property of cleavage not possessed by other building stones, and affects many other qualities as well.

The term "weathering" as applied by geologists to slate where the rock outcrops at the surface of the ground connotes a much more far-reaching change than is considered in this paper. The rock is attacked by the acids of the air in solution in rain water, its cleavage planes are separated by frost action or alternate cooling and heating, and eventually little remains but a yellow, brown, or red clay. But such changes or even their beginnings require thousands, indeed probably millions of years, and can be wholly ignored in this paper. Long before such changes in slate in a building would have become conspicuous, the rest of the building would have suffered serious decay and become uninhabitable. The oldest roofing slate in America has only been exposed for 190 years and yet the buildings on which it was successively placed have been twice torn down. The oldest European slates tested for resistance to weathering are less than 650 years old. Hence weathering effects which are not appreciable in periods of less than 700 years might well be ignored in this paper.

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*Physical Changes:*

Physical changes induced by weathering are those due to changes in temperature—due either to differential expansion of the rock accompanied by flaking or spalling, or to the prizing effect of contained water when it expands on freezing. As to the former, qualitative tests indicate that heat conductivity in slate is far greater in the plane of the cleavage than at right angles thereto; this suggests that roofing slate should spall on rapidly alternating heating and cooling, yet such spalling is only known to occur very rarely, due no doubt in part to the thinness of each individual slate as used. It might be desirable, however, to study the thermal conductivity and coefficient of expansion of slate at right angles and parallel to the cleavage.

As to physical weathering due to frost action, this is affected by porosity, absorption, and permeability. Porosity alone is not the key to weathering qualities, for a rock may bear pores so small that water cannot enter them at any ponderable rate.[1] Experimentally, this difference between porosity and permeability has been demonstrated and emphasized by Kessler and Sligh.[2] Now in slate there is distinctly a direction of greatest and one of least permeability. The cleavage surface has the lowest and sections at right angles to the cleavage have considerably higher absorption and permeability values, as can be shown by qualitative experiments and as has been well recognized by Hirschwald,[3] who directs that before immersing, the test piece be carefully coated with asphalt on all surfaces transverse to the cleavage. The fact enters also into the placing of slate blocks, especially where used in walls, stairs, and the like; here horizontal surfaces, which might hold water, should preferably be the non-absorbing cleavage planes, rather than surfaces cut or broken inclined to the cleavage. Even a smoothly planed surface, if inclined to the cleavage, is more prone to absorb water and should therefore be exposed with judgment.

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[1] G. F. Loughlin, "Indiana Colitic Limestone—Relation of Its Natural Features to Its Commercial Grading," *U. S. Geological Survey Bulletin No. 811-C*, p. 170 (1929).

[2] D. W. Kessler and W. H. Sligh, "Physical Properties of the Principal Commercial Limestones Used for Building Construction in the United States," Bureau of Standards *Technologic Paper No. 349*, p. 534 (1927).

[3] J. Hirschwald, "Handbuch der bautechnischen Gesteinsprüfung," p. 620, Gebrüder Borntraeger, Berlin, (1912).

The author has made a serious attempt to analyze differences in resistance to water ingress with the help of the polarizing microscope. Theoretically, with more nearly parallel mineral particles there should go greater compression, lessened porosity, and hence lower absorbent capacity in the slate. Hence it was thought that the orientation of mineral particles could be used as an index of porosity. An entire paper could be written on this subject alone, but the results so far attained, briefly stated, suggest that the measurement of orientation in mineral particles by the method of Schmidt[1] offers at least a hope for approximate comparison of different slates.

Interest in these facts obviously hinges upon the question of the prizing effect of water in the pores. To this subject Hirschwald has made a noteworthy contribution; he has shown that, from the theoretical considerations, the pore space in rocks will not be completely filled with ice after the contained water freezes unless the ratio between the volume of water absorbed upon slow immersion and the total porosity (as measured by water absorbed on prolonged immersion) is greater than 0.9. Frost action is not dangerous in rocks with lower values of this ratio. In slates known to be highly resistant to frost action, Hirschwald by actual tests finds this ratio to be exceptionally low.[2]

Finally, the microscope may come into play in one more way for studying the resistance of slate to temperature changes. It is known from experience that preexisting fractures in the slate block, rare though they are and perhaps inconspicuous or even invisible to the naked eye, may develop into prominence and cause loss by breakage upon abrupt or irregular heating. Such planes of weakness can be detected with the microscope. Of course, it is true that each individual block to be used cannot be gone over in this way. However, a rough "factor of safety" may be introduced. If planes of weakness, such as "healed" joints, are found to be common in a few samples, the presumption is justified that the rock as a whole may contain them, and hence that they must be carefully searched for before the product of that individual quarry can be used with safety. Illustrations of such planes of weakness are to be found in the various works on slate.[3]

When all is said, a good slate should not and will not show serious signs of decay under frost action, to judge by experiments of the U. S. Bureau of Standards. This fact should be emphasized most strongly and it should be clearly understood that the preceding statements apply only when the use is extended over very long time periods or in places where the slate is very frequently wet and chilled to freezing.

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[1] W. Schmidt, "Statistische Methode beim Gefügestudium kristalliner Schiefer," Sitzungs-Bericht, Wiener Akadamie der Wissenschaft, Vol. 126 (1917).

[2] J. Hirschwald, "Handbuch der bautechnischen Gesteinsprüfung," p. 207, Gebrüder Borntraeger, Berlin, (1912).

[3] T. N. Dale, "Slate in the United States," U. S. Geological Survey *Bulletin No. 586*, Plates VII and VIII (1914).

One other factor affecting weathering is that of abrasion. Not all slates abrade evenly. Where, as in flagging, the effect of constant physical impact wears away the slate, differential abrasion may be effective. A depression is produced, followed commonly by exfoliation, the planes of separation being parallel to the cleavage. This is actually shown in some places where non-uniform ("ribboned") slate has been used for sidewalks. Where the slate is uniform, such unusual "weathering" is not observed. Here again the degree of perfection in the parallelism of mineral constituents, as determined with the petrographic microscope, furnishes a rough measure of the probable resistance to exfoliation. A subcommittee of the Society's Committee D-16 on Slate is at present studying abrasive hardness; it will perhaps shed light on variations in ease of abrasion with direction and thus also indirectly upon one phase of weathering.

### *Chemical Changes:*

Turning now to chemical changes produced in weathering, it may be remarked at the outset that slate is made up of mineral matter resulting from prolonged natural weathering. This mineral matter has been greatly altered by subsequent compression, yet it still remains generally very highly resistant to chemical action, resembling in this respect some sandstones and most quartzites, which, however, are not as readily dressed.

Nevertheless, chemical decay in some slates does take place. We speak here not of the slow oxidation of pyrite to limonite, but of certain other changes which in certain slates become noticeable in periods of 40 or 50 years. The slates with lowest resistance are those that bear calcium, magnesium, or iron carbonates, or their isomorphous mixtures, because it is the carbonates especially that are subject to chemical attack. In order to determine the presence of such carbonates three methods are available: (1) the slate may be subjected to "accelerated" corrosion with carbonic or other acids in unnaturally strong solutions acting for short periods; (2) it may be analyzed chemically; or (3) it may be examined with the polarizing microscope. Time is a factor affecting choice of tests. The corrosion test requires at least a day, and preferably much longer—say, nine months. A single chemical analysis calls for several hours. A thorough microscopic examination by a trained worker is a matter of two hours at most, including the preparation of the specimen for examination; moreover, the test may be made quantitative without more than doubling the time mentioned.[1] However, with increasing numbers of tests, the time factor favoring the microscopic method declines. The rate of carbonate solution is known to vary with the ratio between iron, magnesia, and lime,[2] the last-named being most susceptible. Since in isomorphous mixtures this ratio cannot be satisfactorily determined with the microscope, a chemical analysis is a helpful adjunct.

The manner in which such weathering is brought about is partly chemical, partly physical. When slates bear appreciable amounts of calcium carbonate, dilute solutions of atmospheric carbon dioxide slowly dissolve out the lime salt. In addition, and probably more serious, is the fact that dilute solutions of sulfuric acid also attack the carbonate, forming calcium sulfate. This takes place chiefly under the "lap" of the overlying slate. Thus the locus of the carbonate grain is dissolved, with the beginning of a depression or even of a hole. The calcium sulfate may be carried off in solution or it may be deposited. If the latter, the expansion of the newly-formed gypsum crystals sets up internal pressures and the slate is slowly spalled off in fine chips. Laboratory experiments indicate that this formation of hydrated calcium sulfate does not occur when the slate is continually wet,[3] probably because the newly formed salt is kept in solution. The rate of alteration is determined by (1) porosity, because this affects the accessibility of the slate to the corroding solutions, (2) carbonate content, for reasons already mentioned, and (3) pyrite content, because pyrite upon oxidizing, however slowly, contributes sulfuric acid.

Slate is subjected to especially severe "artificial weathering" conditions when used in toilet stalls, chemical laboratory sinks, and the like, where exposed to strong acids and alkalies. Here again it proves very resistant. Lancaster and Behre,[4] following earlier workers, found that, of four solutions of equal molar strength containing sulfuric and hydrochloric acids and sodium and ammonium hydroxides, respectively, sulfuric acid was by far the strongest agent, again partly because of the formation of gypsum. It was also shown that solutions of alkalies have practically no color or other chemical effect on slate; the action of acids is from ten to twenty times as strong. As "weathering" agents, whether natural or artificial, alkalies may therefore be ignored in future experiments.

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[1] Albert Johannsen, "A Planimeter Method for the Determination of the Percentage Composition of Rocks," *Journal of Geology*, Vol. 27, p. 276 (1919).

[2] T. N. Dale, "Slate in the United States," *U. S. Geological Survey Bulletin No. 586*, p. 55 (1914).

[3] "Destructive Weathering of Roofing Slate," Bureau of Standards *Technical News Bulletin No. 164*, p. 121 (1930).

[4] C. L. Lancaster and C. H. Behre, Jr., "Chemical Experiments on Acid and Alkali Resistance of Slates," *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part I, p. 889 (1930).

Far more important for practical purposes are the luster and color changes that take place in slate with the passage of time. The subject cannot be discussed exhaustively here. To date very little progress has been made in the direction of practical tests of these changes. Virtually all slates lose some of their initial luster upon prolonged exposure. Empirically it is known that the higher the quartz content, the more nearly permanent is the sheen, probably again because of the correspondingly small amount of the carbonates which would be the first components altered.

Color changes are of two sorts, those that are slow and those that, as in certain slates, take place in a few years—say, five or ten. For both of these changes, since the term "weathering" implies a disintegration which here does not obtain, the term "color-aging" has been suggested. One of the first needs in studying color changes is a suitable scale that can be used, if not for commercial grading and specifications, at least for scientific testing. Certain green slates especially show a rapid change and hence have been designated "fading" slates; in this case the color change can be measured with the Ridgway color chart or, less satisfactorily, with its modified form as recently published.[1] By the Ridgway system, varying hues and shades are designated by a number and letter—a plan far superior, for scientific purposes, to that of descriptive terms, such as blue-gray or purple. Thus, the weathering change in blue-gray slate from Northampton County, Pa., is from 15$^4$m when fresh to 15$^4$k. This accurate nomenclature, however, could not wisely be extended to commercial specifications as any rock—whether slate, marble, or granite—is heterogeneous and normally shows marked variations between extremes.

Although some of the pyrite in slates is seen to undergo rusting through natural weathering over a very long period—say, several thousands of years—color changes that take place in the ordinary life and use of roofing or structural slate are probably due wholly to the alteration of ferrous carbonate, as maintained by W. F. Hillebrand.[2] The iron carbonate is attacked by acids with the formation of soluble ferric salts and, by hydrolysis, of ferric hydroxides; the latter may then coat the slate surfaces, especially on the more carbonaceous areas, as demonstrated experimentally by Lancaster and Behre.[3]

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[1] Robert Ridgway, "Color Standards and Color Nomenclature," Washington, D. C. (1912); also, M. I. Goldman and H. E. Merwin, "Color Chart for the Description of Sedimentary Rocks," National Research Council, Washington, D. C. (1928).

[2] T. N. Dale, "Slate in the United States," U. S. Geological Survey *Bulletin No. 586*, p. 55 (1914).

[3] C. L. Lancaster and C. H. Behre, Jr., "Chemical Experiments on Acid and Alkali Resistance of Slates," *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part I, p. 889 (1930).

In the case of "color-aging," the microscopic and chemical analyses are of presumptive value only. The sole really dependable criterion for color permanence is actual chemical testing along the lines already laid down, recognizing the validity of acceleration by using reagents of unnaturally high concentrations. It would seem decidedly worth while to devise a series of tests that will effectively serve this purpose. To do so will be breaking ground in a wholly new field of research.

The author, again, would not wish to create any possible misapprehension. Slate is a remarkably resistant mineral product and what has been said is given not with the intent of leaving an impression to the contrary, but to indicate some directions which future research may take for a better understanding of its properties.

# WEATHERING OF AGGREGATES

By Loring O. Hanson[1]

## Synopsis

The main object of this investigation was to determine the durability of various crushed rocks and mortars and to develop an accelerated test to measure their resistance to weathering. A freezing and thawing test was used to duplicate weathering and this test was paralleled by a series of sodium sulfate tests and absorption tests.

Twenty-four coarse aggregates from Wisconsin were represented by artificially graded samples of crushed material and also by 2 by 2-in. cylindrical cores drilled from the rock. The change in the fineness modulus of the graded aggregate and the loss of strength of the cores furnished evidence of the amount of disintegration produced by these tests. In addition, a number of mortar cylinders and prisms were made from four types of sand using several consistencies and mixes. Part of these specimens were broken before and part after durability tests.

The disintegration of the rock caused by the freezing-and-thawing test was in agreement with what might have been expected from field inspection of weathered material at the quarry site. One of the sodium sulfate tests employed proved quite destructive to the rocks and mortars, causing a degree of disintegration roughly comparable to that occasioned by the freezing-and-thawing test. No relation was found to exist between the durability of the rock and any of its absorptive properties.

## Introduction

This paper is a progress report on work now being carried on at the Materials Laboratory of the University of Wisconsin to determine the weathering characteristics of certain rocks and mortars and to develop and investigate accelerated weathering tests, using sodium sulfate as the destructive agent. In addition, the absorptive properties of the various materials are being determined. The work, which has been under way for about one year, is being carried out by the author under the general supervision of Professor M. O. Withey of the Department of Mechanics. Messrs. L. M. Basford, W. R. Hicks, R. H. Kehl, and M. J. Scott assisted in the actual experimental work. Samples of the rocks to be tested were procured by the Wisconsin State Highway Department.

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## WEATHERING TESTS ON ROCKS

*Materials:*

The rocks subjected to the various tests are listed in Table I. With the exception of Janesville gravel, these rocks were separately crushed in the laboratory from samples taken by engineers of the State Highway Department. The samples were taken directly from the quarry faces, consisting partly of large pieces from which cores were to be drilled and partly of smaller pieces. Record was kept of the quarry represented and the manner in which the material seemed to be "weathering" at the site.

The limestones of southern and central Wisconsin are broadly grouped under four heads, each being represented in these tests by a number of samples. The Niagara is a dolomitic limestone of relatively fine texture, free from small irregular cavities and fairly uniform in quality due to massive bedding. It is hard, has a low percentage of wear, has a smooth fracture and breaks into roughly rectangular pieces of uneven dimensions. The Lower Magnesian limestone is somewhat irregular in character, sometimes being cherty and sandy with varying grain size. It is quite hard and wear resisting and breaks into blocks with hackly surfaces but more nearly of equal dimensions than the Niagara. The Galena is a magnesian limestone lacking uniformity in character, being either thin or massive bedded. It is coarsely crystalline and sometimes contains chert. It is of ordinary hardness and wear resistance and breaks into blocks of irregular outline and rough surface. The Trenton is an argillaceous magnesian limestone which is quite thin bedded, the parting planes due to clay sometimes being very close together. It is low in hardness and resistance to wear. Frequently it contains some "Glass-rock," which is a nearly pure dark-colored limestone breaking with a conchoidal fracture.

Janesville gravel is quite largely of Trenton derivation with the poorer material weathered away. The Potsdam sandstone is soft and friable, consisting of quartz grains weakly cemented together. The Wisconsin granite is a relatively coarse-grained granite.

Of the limestones, the Trenton is, in general, the least resistant to weathering. This rock tends to separate along clay bedding planes into very small fragments, and consequently quarries which have not been worked for some years often show much disintegration of the surface material. In quarries worked at present, chunks thrown some distance away by earlier shots will be found broken up by weathering. Others, which appear solid enough, can be kicked into pieces with the foot. However, this condition is not equally noticeable at all quarries of this limestone.

Occasionally a Galena limestone shows a very considerable degree of weathering at the quarry site, and less frequently, a Lower Magnesian limestone is badly weathered. With the occasional exception of certain beds, most of the Niagara limestone on quarry faces shows very little weathering.

A number of cylinders representing each of the general types of limestone were drilled from the samples sent to the laboratory. These are listed in Table III. A soft steel pipe of slightly over 2 in. inside diameter and about 4 in. long, with a shallow slot cut down the outside, was threaded to a drill chuck. With Ottawa sand as an abrasive, this tool was used to drill the cylinders which were then cut down on a lathe to a length of 2 in. The drilling was perpendicular to the bedding planes when such planes were in evidence. The entire operation gave a very satisfactory specimen, approximately 2 by 2 in. in dimensions, and with the ends sufficiently plane that any cylinder could be broken in the testing machine with no bedding.

The remainder of each sample was crushed in the laboratory to a maximum size of 1½ in. and sieved over a ⅜-in. sieve to remove all dust. Individual samples for the various tests were then made up as follows: 1200 g. retained on a 1-in. sieve, 600 g. passing a 1-in. and retained on a ¾-in. sieve, and 600 g. passing a ¾-in. and retained on a ⅜-in. sieve. Based on percentages retained on 1-in., ¾-in., ⅜-in., and No. 4 sieves, the above grading gives a fineness modulus of 3.25.

*Freezing-and-Thawing Test:*

One sample of each of the crushed rocks and three of each kind of rock cylinders were subjected to the freezing-and-thawing test. The crushed rock samples were dried at 212° F. for 4 hours, immersed in water at about 60° F. for 24 hours, and then placed in the freezer, individual samples being covered with water. After 16 hours in the freezer, the samples were removed and thawed in tap water at about 60° F. for 8 hours, then again placed in the freezer. Each 24-hour cycle constituted one reversal. The rock was dried in the oven only when a sieve analysis was made. The rock cylinders were subjected to much the same treatment with the exception that while in the freezer they were stood on end in shallow trays with about ½ in. of water in the bottom and were not dried in the oven after the first immersion. Temperatures in the freezer, which was cooled by the expansion of ammonia, ranged from 9 to 22° F. with an average of about 14° F. When it was not convenient to make a reversal every 24 hours, the material was generally left in the freezer.

*Sodium Sulfate Tests:*

The samples of crushed rock were subjected to the following tests using sodium sulfate as a destructive agent. Elsewhere in the report these tests are listed only by number. In the following descriptions of the tests, oven-dried refers to drying in a preheated oven at a temperature of from 212 to 221° F. and air-dried refers to drying in air at a temperature of approximately 70° F. Hot sodium sulfate refers to immersion in a concentrated solution at 115 to 130° F. and cold sodium sulfate to immersion in a concentrated solution at 70° F.

TABLE I.—WEATHERING TESTS ON CRUSHED ROCK.

Each value represents one sample of 2400 g., artificially graded. Fineness modulus based on percentages retained on 1-in., ¾-in., ⅜-in., and No. 4 sieves.

| Material | Fineness Modulus | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Original | After Freezing and Thawing | | After Subjection to Sodium Sulfate Tests | | | | |
| | | 80 Reversals | 140 Reversals | Test No. 1 | Test No. 2 | Test No. 2 (Re-run) | Test No. 3 | Test No. 4 |
| Galena Limestone: | | | | | | | | |
| Kieler | 3.25 | 3.05 | 3.00 | 3.27 | 3.13 | 3.15 | 3.24 | 3.22 |
| Yonke | 3.25 | 2.63 | 2.53 | 3.25 | 3.08 | .... | 3.23 | .... |
| Granite: | | | | | | | | |
| Wisconsin | 3.25 | Lost | Lost | 3.23 | 3.24 | 3.20 | 3.28 | 3.23 |
| Gravel: | | | | | | | | |
| Janesville | 3.25 | 2.87 | 2.80 | 3.26 | 3.16 | 3.13 | 3.22 | 3.17 |
| Lower Magnesian Limestone: | | | | | | | | |
| Bashford Hill | 3.25 | 3.21 | 3.17 | 3.25 | 3.20 | 3.21 | 3.22 | 3.21 |
| Cox | 3.25 | 3.13 | 3.11 | 3.26 | 3.14 | 3.17 | 3.23 | 3.19 |
| Elmwood | 3.25 | 3.16 | 3.11 | 3.25 | 3.23 | 3.19 | 3.21 | 3.17 |
| Grandad Bluff | 3.25 | 2.68 | 2.57 | 3.24 | 3.13 | 3.07 | 3.18 | 3.10 |
| Point Jude | 3.25 | 3.15 | 3.14 | 3.28 | 3.16 | .... | .... | .... |
| Readfield | 3.25 | 3.22 | 3.16 | 3.27 | 3.17 | 3.23 | 3.23 | 3.23 |
| Tulley | 3.25 | Lost | 3.15[a] | 3.26 | 3.18 | 3.01 | 3.27 | 2.97 |
| Niagara Limestone: | | | | | | | | |
| Consumers | 3.25 | 3.24 | 3.21 | 3.23 | 3.16 | 3.23 | 3.23 | 3.23 |
| Mayville | 3.25 | Lost | 3.16[a] | 3.23 | 3.10 | 3.04 | 3.24 | 3.11 |
| Nasbra | 3.25 | Lost | .... | 3.30 | 3.22 | 3.23 | 3.25 | 3.23 |
| Sturgeon Bay | 3.25 | 3.20 | 3.09 | 3.21 | 3.21 | 3.11 | 3.23 | 3.23 |
| Waukesha | 3.25 | 2.89 | 2.86 | 3.25 | 3.10 | 3.15 | 3.22 | 3.07 |
| Sandstone (Potsdam): | | | | | | | | |
| Dunnville | 3.25 | 3.11 | 2.87 | 3.26 | 3.21 | 3.22 | 3.21 | 2.80 |
| Trenton Limestone: | | | | | | | | |
| Critchlow | 3.25 | 1.83 | 1.68 | 3.19 | 2.48 | 2.50 | 2.81 | 2.95 |
| Draves | 3.25 | 2.16 | 2.13 | 3.21 | 2.52 | 2.23 | 2.93 | 2.89 |
| Duck Creek | 3.25 | 2.80 | 2.69 | 3.22 | 2.93 | 2.90 | 3.19 | 3.12 |
| Mau | 3.25 | 2.35 | 2.22 | 3.22 | 3.05 | 2.50 | 3.10 | 2.66 |
| Mineral Point | 3.25 | 2.23 | 2.01 | 3.24 | 2.91 | 2.79 | 2.96 | 2.97 |
| Sherry | 3.25 | 2.40 | 2.32 | 3.19 | 2.97 | 2.88 | 3.06 | 2.91 |
| York Church | 3.25 | Lost | 2.35[a] | 2.37 | 2.02 | 2.22 | 2.59 | .... |

[a] 60 Reversals.

*Sodium Sulfate Test No. 1.*—Oven-dried 4 hours, hot sodium sulfate 16 hours, oven-dried 4 hours, and then alternated cold sodium sulfate and hot sodium sulfate at two-hour intervals until ten reversals were reached. Oven-dried 4 hours and sieved without washing. Allowed to remain in either hot or cold sodium sulfate overnight.

*Sodium Sulfate Test No. 2.*—Oven-dried 4 hours, hot sodium sulfate 16 hours or longer, oven-dried 4 hours, hot sodium sulfate 4 hours or longer, cold sodium sulfate 4 hours, oven dried 4 hours, and the last three repeated until 5 reversals were reached, then sieved without washing. Allowed to remain in hot sodium sulfate overnight.

*Sodium Sulfate Test No. 3.*—Material in this test was air-dried from laboratory storage. Five cycles of 8 hours immersion in hot sodium sulfate and 16 hours air-dried. Allowed to become thoroughly dry after last cycle and sieved unwashed.

TABLE II.—WEATHERING TESTS ON CRUSHED ROCK.

Each value represents one sample of 2400 g., artificially graded.

| Material | Percentage of Material Passing $\frac{3}{8}$-in. Sieve | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Original | After Freezing and Thawing | | After Subjection to Sodium Sulfate Tests | | | | |
| | | 80 Reversals | 140 Reversals | Test No. 1 | Test No. 2 | Test No. 2 (Re-run) | Test No. 3 | Test No. 4 |
| Galena Limestone: | | | | | | | | |
| Kieler | 0.0 | 5.9 | 6.5 | 0.6 | 2.8 | 2.2 | 0.5 | 0.7 |
| Yonke | 0.0 | 15.1 | 17.9 | 0.3 | 3.1 | .... | 1.9 | .... |
| Granite: | | | | | | | | |
| Wisconsin | 0.0 | Lost | Lost | 0.3 | 0.5 | 0.2 | 0.0 | 0.4 |
| Gravel: | | | | | | | | |
| Janesville | 0.0 | 8.5 | 10.7 | 0.1 | 2.1 | 3.0 | 1.0 | 1.2 |
| Lower Magnesian Limestone: | | | | | | | | |
| Bashford Hill | 0.0 | 2.6 | 3.5 | 0.4 | 1.1 | 0.6 | 0.5 | 1.2 |
| Cox | 0.0 | 3.8 | 4.5 | 0.3 | 2.7 | 0.8 | 0.8 | 1.5 |
| Elmwood | 0.0 | 1.4 | 2.2 | 0.4 | 1.0 | 0.8 | 0.5 | 2.2 |
| Grandad Bluff | 0.0 | 15.1 | 17.2 | 0.0 | 1.5 | 5.0 | 1.6 | 4.0 |
| Point Jude | 0.0 | 4.3 | 5.0 | 0.3 | 1.7 | ... | ... | ... |
| Readfield | 0.0 | 1.1 | 1.6 | 0.3 | 0.8 | 1.1 | 1.1 | 0.4 |
| Tulley | 0.0 | Lost | 1.1[a] | 0.4 | 1.5 | 5.1 | 0.2 | 9.0 |
| Niagara Limestone: | | | | | | | | |
| Consumers | 0.0 | 0.9 | 1.1 | 0.7 | 1.0 | 0.2 | 0.8 | 0.6 |
| Mayville | 0.0 | Lost | 2.5[a] | 0.7 | 3.6 | 4.2 | 0.8 | 5.1 |
| Nasbra | 0.0 | Lost | Lost | 0.2 | 1.2 | 0.5 | 0.7 | 0.3 |
| Sturgeon Bay | 0.0 | 3.1 | 4.5 | 0.5 | 2.4 | 2.7 | 0.7 | 0.9 |
| Waukesha | 0.0 | 9.3 | 10.1 | 0.7 | 5.3 | 3.6 | 1.2 | 5.7 |
| Sandstone (Potsdam): | | | | | | | | |
| Dunnville | 0.0 | 4.8 | 11.4 | 0.4 | 0.6 | 0.9 | 0.7 | 14.5 |
| Trenton Limestone: | | | | | | | | |
| Critchlow | 0.0 | 33.6 | 38.8 | 1.2 | 19.1 | 23.1 | 15.0 | 7.8 |
| Draves | 0.0 | 25.5 | 27.2 | 1.1 | 14.5 | 26.7 | 11.3 | 10.5 |
| Duck Creek | 0.0 | 11.3 | 13.1 | 1.1 | 8.9 | 7.3 | 2.4 | 2.6 |
| Mau | 0.0 | 26.6 | 30.9 | 0.4 | 7.0 | 20.6 | 6.1 | 13.7 |
| Mineral Point | 0.0 | 30.8 | 35.0 | 0.6 | 7.8 | 13.6 | 8.2 | 11.0 |
| Sherry | 0.0 | 18.6 | 21.1 | 0.9 | 7.2 | 7.7 | 5.1 | 6.6 |
| York Church | 0.0 | Lost | 26.4[a] | 24.8 | 33.0 | 30.5 | 20.0 | ... |

[a] 60 Reversals.

*Sodium Sulfate Test No. 4.*—Oven-dried 4 hours, hot sodium sulfate 16 hours or longer, oven-dried 4 hours, air-dried 16 hours, hot sodium sulfate 4 hours with the last three repeated until 5 reversals were reached. Oven-dried 4 hours. This material was then washed 6 times in water at about 122° F. in as many days, oven-dried to remove moisture, and a sieve analysis made.

The rock cylinders were subjected only to test No. 2.

The tanks containing the solution of sodium sulfate at 70° F. were kept in a water-spray moist closet. The "hot" solution was kept in a small tank in turn contained in a larger tank nearly filled with water, bathing the sides of the smaller tank. Heating units and thermostatic control were used, the heating units being immersed in the outer tank. The large reserve of heat carried by the water in the outer tank effectively served to keep the temperature of the inner tank constant.

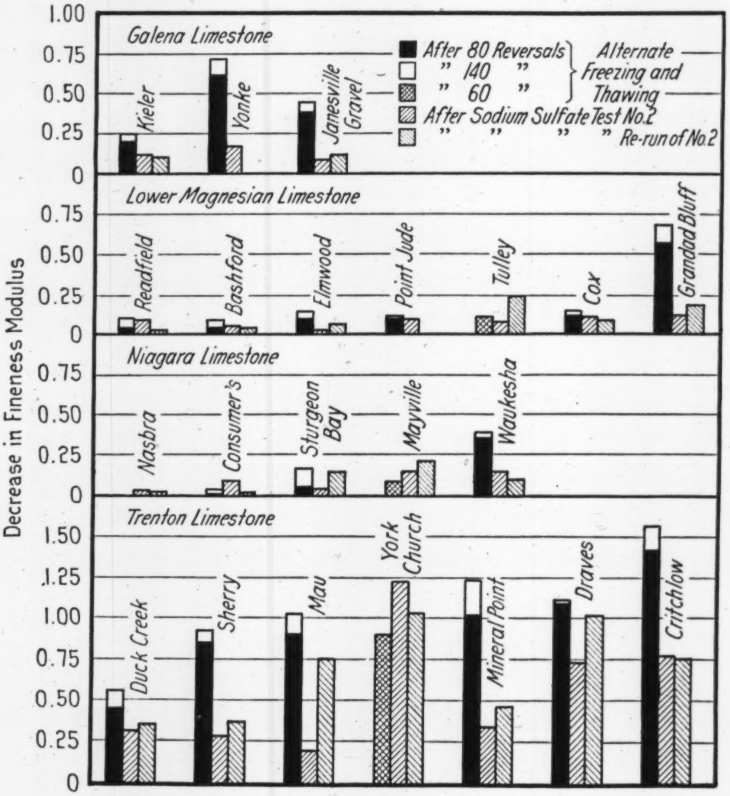

Fig. 1.—Results of Weathering Tests on Rocks.

*Absorption Tests:*

Absorption determinations were made for all of the rocks. Each sample weighed approximately 1000 g. and consisted of particles about 1 in. in greatest dimension. The material was dried for 72 hours at 167° F. in an electric oven, immersed in water, and surface dried after the necessary interval with coarse rags. From two to five determinations were made. Tests on samples from three of the general types of limestone showed that there was no appreciable loss in weight after the 72 hours and that subsequent absorption was the same as if a drying temperature of 212° F. had been used. The drying temperature of 167° F. was used in drying mortar specimens for the absorption test in order to avoid any possible destructive effects of drying at a temperature above the boiling point of water. The same drying temperature was used for the rock to enable comparison of test data and to avoid frequent adjustments of oven temperature.

*Results of Tests:*

The relative destructive effect of the freezing-and-thawing test and the various sodium sulfate tests employed on the crushed rocks is shown in Tables I and II. A comparison is also made between the freezing-and-thawing test and the results from the two runs of sodium sulfate test No. 2 in Fig. 1.

The first point of interest lies in the fact that the first 80 reversals of freezing and thawing were much more destructive to the rock than the last 60. The original plan was to run a mechanical analysis at 100 reversals but an accident to the equipment made it necessary to carry this out at 80 cycles. In many ways it would be very desirable to have a complete history of the disintegration with a mechanical analysis every five or ten reversals but this introduces as an additional unknown factor the destructive effect of heating the rock to high temperatures so many times. Another point worthy of investigation would be the effect of placing the saturated rock in the freezer without immersion. Inasmuch as both methods could not be used, the continuous immersion in individual containers was used in these tests as it prevented the washing away of any fine material.

A wide variation was found in the destructive effect of the various sodium sulfate tests. Sodium sulfate exists in solution with water as an anhydrous salt at temperatures above 90° F. At a temperature of 70° F. it is in the form of a hydrate with a consequent change in volume. Furthermore, approximately twice as much salt is soluble in the water at the higher temperature. To get the maximum destructive effect, advantage should be taken of both of these conditions.

Test No. 1, which consisted essentially of changing the sample from hot to cold solution for a period of ten cycles, did not, in most instances, have any considerable destructive effect. Test No. 3, which consisted essentially of changes from hot solution to air for five cycles, caused greater destruction than No. 1 but was not as stringent as test No. 2 or test No. 4. Test No. 4, which consisted of 5 cycles of immersion in hot sodium sulfate solution followed by oven drying and air drying, was next in order. A number of samples were first subjected to this test and a mechanical analysis made without washing the material. It was clearly evident that material which was badly shattered was being held together by crystallized salt, so that it was decided to try the effect of washing the salt out.

TABLE III.—WEATHERING TESTS ON ROCK CYLINDERS

Each value represents average of three specimens unless otherwise noted.

| Material | Mark | Crushing Strength, lb. per sq. in. | | Condition of Specimens | |
|---|---|---|---|---|---|
| | | Untreated | After Subjection to Sodium Sulfate Test No. 2 | After 100 Reversals of Freezing and Thawing | After Subjection to Sodium Sulfate Test No. 2 |
| Galena Limestone: | | | | | |
| Kieler | 8 | 18 300 | 16 500 | Very slight shipping on one specimen. | Outwardly unchanged. |
| Yonke | 4 | 9 200 | 5 300 | One specimen badly chipped and one slightly. | Slight chipping on one specimen. |
| Loser Magnesian Limestone: | | | | | |
| Bashford Hill | 13 | 18 400 | 18 300 | Very slight chipping on one specimen. | Outwardly unchanged. |
| Cox | 7 | 14 600 | 13 500 | Cracking and crumbling of bottom of one specimen. Another chipped. | Outwardly unchanged. |
| Elmwood | 2 | 14 400 | 18 400 | Outwardly unchanged. | Outwardly unchanged. |
| Grandad Bluff | 14 | 17 800 | 13 800 | One specimen badly chipped and one slightly chipped. | Outwardly unchanged. |
| Readfield | 5 | 27 000 | 29 000 | Outwardly unchanged. | Outwardly unchanged. |
| Niagara Limestone: | | | | | |
| Consumers | 1 | 11 300 | 15 500 | Outwardly unchanged. | Outwardly unchanged. |
| Mayville | 11 | 13 100 | 9 200 | Some crumbling on bottom of one specimen. | One specimen badly crumbled. One starting to crack. |
| Nasbra | 3 | 29 700 | 27 000 | Outwardly unchanged. | Outwardly unchanged. |
| Waukesha | 6 | 18 400 | 21 100 | Very slight chipping on one specimen. | Outwardly unchanged. |
| Sandstone (Potsdam): | | | | | |
| Dunnville (4 specimens) | 15 | 2 200 | 3 100 | Some shaling on two specimens. | One specimen starting to crack. |
| Trenton Limestone: | | | | | |
| Critchlow | 10 | 9 300 | 6 800 | One specimen cracked in two and crumbling. | Two specimens entirely broken up. |
| Draves | 9 | 13 400 | 5 200 | One specimen crumbling badly. One chipped on bottom. | Two specimens entirely broken up. One chipped on end. |
| Mau (4 specimens) | 12 | 11 200 | 4 300 | One specimen cracked in two. One starting to crack. | Two specimens entirely broken up. One specimen broken across end. Chipping on end on one. |
| Sherry (4 specimens) | 16 | 16 500 | 11 600 | One specimen disintegrated. One broken in two. | Two specimens broken in two. Chipping on one. |

Six washings in hot water at about 122° F. was sufficient to remove the salts and disintegrated material was free to separate. Consequently, all of the samples in test No. 4 were treated in this manner, it being necessary to re-run some. The results based on this new procedure are incorporated in the tables.

Sodium sulfate test No. 2, consisting of 5 cycles of immersion in hot solution, cold solution and oven drying, was the most destructive of all tests and showed the best correlation with the freezing-and-thawing tests. A study of the relative disintegration caused by the two tests is given in Fig. 1. The re-run of test No. 2 was made as a check to determine how closely experimental values might be duplicated.

The above tests seem to show that oven drying is a necessary part of the sodium sulfate test in order to secure maximum destructive effect. Furthermore, a test such as No. 2, in which oven drying is employed, begins to approach in severity the result of a great many cycles of freezing and thawing. It is believed that more disintegration may be secured by following test No. 2 by washing, and this is being included in further tests.[1] Samples are introduced into the solution in individual containers with small holes on the side to facilitate draining in the present tests. In removing the samples from the tanks, some fine material is likely to be washed away by the draining. Removing the salt together with a final weighing furnishes a check upon this.

TABLE IV.—ABSORPTION TESTS ON CRUSHED ROCK.

Dried in oven at 167° F. for 72 hours previous to immersion in water.

| Material | Absorption of Oven-dry Weight, per cent Immersion in Water for | | | Ratio, 24-hour Cold Absorption to 5-hour Boil Absorption | Relative Resistance to Alternate Freezing and Thawing as Measured by Change in Fineness Modulus After 80 Reversals |
|---|---|---|---|---|---|
| | 6 Hours at 70° F. | 24 Hours at 70° F. | 5 Hours Boiling | | |
| **Galena Limestone:** | | | | | |
| Kieler | 0.83 | 0.89 | 1.45 | 0.61 | 11 |
| Yonke | 2.14 | 2.14 | 3.30 | 0.65 | 16 |
| **Granite:** | | | | | |
| Wisconsin | 0.23 | 0.23 | 0.29 | 0.79 | .. |
| **Gravel:** | | | | | |
| Janesville | 1.81 | 1.93 | 2.12 | 0.91 | 13 |
| **Lower Magnesian Limestone:** | | | | | |
| Bashford Hill | 0.69 | 0.74 | 0.82 | 0.90 | 3 |
| Cox | 1.67 | 1.75 | 2.57 | 0.68 | 9 |
| Elmwood | 1.15 | 1.22 | 2.16 | 0.56 | 5 |
| Grandad Bluff | 2.53 | 2.69 | 4.18 | 0.64 | 15 |
| Point Jude | 1.25 | 1.27 | .... | .... | 6 |
| Readfield | 0.49 | 0.51 | 0.60 | 0.85 | 2 |
| Tulley | 1.67 | 1.82 | 2.38 | 0.76 | 7 |
| **Niagara Limestone:** | | | | | |
| Consumers | 0.95 | 0.99 | 2.23 | 0.44 | 1 |
| Mayville | 0.44 | 0.44 | 1.17 | 0.38 | 8 |
| Nasbra | 0.50 | 0.50 | 0.64 | 0.78 | .. |
| Sturgeon Bay | 0.43 | 0.45 | 0.43 | 1.04 | 4 |
| Waukesha | 0.97 | 1.09 | 1.04 | 1.04 | 12 |
| **Sandstone (Potsdam):** | | | | | |
| Dunnville | 11.53 | 11.77 | 17.00 | 0.69 | 10 |
| **Trenton Limestone:** | | | | | |
| Critchlow | 1.45 | 1.60 | 2.10 | 0.76 | 22 |
| Draves | 1.47 | 1.47 | 1.74 | 0.84 | 20 |
| Duck Creek | 0.68 | 0.73 | 0.66 | 1.10 | 14 |
| Mau | 2.64 | 2.78 | 3.76 | 0.74 | 18 |
| Mineral Point | 1.56 | 1.64 | 2.59 | 0.63 | 19 |
| Sherry | 1.51 | 1.58 | 1.61 | 0.98 | 17 |
| York Church | 3.29 | 3.44 | 3.38 | 1.02 | 21 |

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[1] Experiments made subsequent to the writing of this report show that sodium sulfate test No. 2, followed by washing, has a destructive effect more closely correlated to that of 80 or 140 reversals of freezing and thawing than any of the sodium sulfate tests for which data are included in the report.

Table III shows the effect of sodium sulfate test No. 2 on various rock cylinders. Yonke Galena, Grandad Bluff Lower Magnesian limestone and the four Trenton limestones showed the greatest amount of weakening by this test. An examination of Fig. 1 will show that these same crushed rocks were the most affected by the freezing-and-thawing test. While the crushing strength of the rock cylinders subjected to freezing and thawing has not yet been determined, their general condition at the end of 100 reversals was in good agreement with the condition of other cylinders after subjection to the sodium sulfate test.

Some of the rocks tested showed more of a tendency to disintegrate into extreme fines than others. Because of this and also because a fineness modulus based on a mechanical analysis takes into account what happens to the larger pieces in the sample more than does the statement of the amount of material passing a $\frac{3}{8}$-in. sieve after the tests, the fineness modulus has been stressed as a measure of disintegration.

In concluding the discussion on the weathering tests, it can be said that the freezing-and-thawing test gave results that checked quite satisfactorily with evidence gained by actual inspection of weathered material at the quarry site. The Trenton limestones, followed by the Galena limestones and the one Lower Magnesian, show these same tendencies in the field. The accelerated test, sodium sulfate test No. 2, shows a very fair degree of correlation with the freezing-and-thawing test and it is believed that it can be further improved upon. It is advisable that larger samples than were available for these tests be used in the future. A sample from two to four times as large as was used here should prove more satisfactory.

Table IV shows the results of the absorption tests on the various rocks. Study fails to show any relationship between their absorption, rate of absorption, ratio of cold absorption to boiling absorption, and their resistance to weathering. This applies not only for the entire list of specimens but also within any one class.

## TESTS ON MORTARS

A number of mortar specimens were subjected to tests with sodium sulfate as the destructive agent and others are being tested by alternate freezing and thawing. An outline is given in Table V.

### *Mortars:*

Four sands were used in making the specimens. Janesville sand, with a fineness modulus of 2.7, contains approximately 65 per cent of quartz with dolomite the other principal constituent. Maribel sand, with a fineness modulus of 3.3, contains a good many large rounded particles. It is largely dolomitic with some quartz in the fine material. Winona is a harsh sand containing quartz, trap and some feldspar, and has a fineness modulus of 3.0. The sample of Wisconsin Michigan Power sand came from a pit from which mixed aggregate had given unsatisfactory results on a piece of construction exposed to the weather. This sand had a fineness modulus of 2.2 and was largely composed of fine quartz grains. The sample used contained 4 per cent of silt but passed the colorimetric test.

TABLE V.—TESTS ON MORTARS.

Each value represents three specimens.

| Mark | Sand | Mix | Consistency as Measured by Flow Table | Water-Cement Ratio by Volume | Compressive Strength of 2 by 4-in. Cylinders, lb. per sq. in. | | | | | Modulus of Rupture for 1 by 1 by 6-in. Cross-Bending Specimens, lb. per sq. in. | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | After 1 Day in Air and 27 Days in Water | After 1 Day in Air, 27 Days in Water, and Subsequent Subjection to Sodium Sulfate Test No. 2[a] | After 1 Day in Air, 27 Days in Water, and Subsequent Subjection to Sodium Sulfate Test No. 4[a] | Condition of Specimens in Freezer After 80 Reversals of Freezing and Thawing[b] | Ratio of Strength After Sodium Sulfate Test No. 2 to 28-day Compressive Strength | After 1 Day in Air and 27 Days in Water | After 1 Day in Air, 27 Days in Water, and Subsequent Subjection to Sodium Sulfate Test No. 2[a] | After 1 Day in Air, 27 Days in Water, and Subsequent Subjection to Sodium Sulfate Test No. 4[a] | Condition of Specimens in Freezer After 80 Reversals of Freezing and Thawing[b] | Ratio of Strength After Sodium Sulfate Test No. 2 to 28-day Compressive Strength |
| J-1 | Janesville.. | 1:2 | Dry | 0.454 | 5700 | 5170 | 6290 | 2 | 0.91 | 1530 | 1450 | 1260 | 2 | 0.95 |
| J-2 | | 1:2 | Medium | 0.515 | 8130 | 8450 | 8710 | 2 | 1.04 | 1600 | 1610 | 1630 | 1 | 1.01 |
| J-3 | | 1:2 | Wet | 0.567 | 6930 | 6080 | 5760 | 1 | 0.88 | 1530 | 1680 | 1390 | 2 | 1.10 |
| J-4 | | 1:3 | Dry | 0.584 | 4980 | 2700 | 5980 | 2 | 0.54 | 1210 | 1010 | 1300 | 3 | 0.83 |
| J-5 | | 1:3 | Medium | 0.666 | 5740 | 3570 | 6050 | 2 | 0.62 | 1150 | 1180 | 1180 | 3 | 1.03 |
| J-6 | | 1:3 | Wet | 0.745 | 5410 | 3790 | 5820 | 2 | 0.70 | 1020 | 1090 | 1020 | 1 | 1.07 |
| J-7 | | 1:5 | Dry | 0.882 | 2810 | 1750 | 2970 | 3 | 0.62 | 700 | 660 | 550 | 2 | 0.94 |
| J-8 | | 1:5 | Medium | 1.08 | 2750 | 1600 | 3110 | 3 | 0.58 | 705 | 460 | 360 | 4 | 0.65 |
| J-9 | | 1:5 | Wet | 1.29 | 2580 | 1470 | 2370 | 5 | 0.57 | 570 | 0 | 430 | 4 | 0.0 |
| P-1 | Wisconsin-Michigan Power Co. | 1:2 | Medium | 0.526 | 5820 | 5620 | 6300 | 2 | 0.97 | 1110 | 1030 | 1270 | 1 | 0.93 |
| P-2 | | 1:3 | " | 0.667 | 4030 | 1940 | 4090 | 2 | 0.48 | 990 | 690 | 920 | 2 | 0.70 |
| P-3 | | 1:5 | " | 0.976 | 2020 | 1340 | 2450 | 2 | 0.66 | 505 | 240 | 530 | 2 | 0.47 |
| M-1 | Maribel... | 1:2 | " | 0.475 | 8100 | 8660 | 9750 | 1 | 1.07 | 1630 | 1990 | 2010 | 1 | 1.22 |
| M-2 | | 1:3 | " | 0.620 | 6500 | 6140 | 6120 | 2 | 0.94 | 1290 | 1210 | 1490 | 1 | 0.94 |
| M-3 | | 1:5 | " | 1.00 | 3130 | 2010 | 2720 | 3 | 0.64 | 785 | 400 | 690 | 4 | 0.51 |
| W-1 | Winona... | 1:2 | " | 0.484 | 7490 | 5840 | 8320 | 1 | 0.78 | 1390 | 1550 | 1490 | 1 | 1.12 |
| W-2 | | 1:3 | " | 0.650 | 6000 | 2750 | 4860 | 2 | 0.46 | 995 | 920 | 920 | 2 | 0.92 |
| W-3 | | 1:5 | " | 1.09 | 2050 | 780 | 1860 | 4 | 0.38 | 515 | 0 | 370 | 3 | 0.0 |

[a] Immersed in water 2 days after completion of sodium sulfate test. Age at breaking, 37 days.

[b] Numbers refer to relative degree of disintegration; 1 means no effect noticeable; 5 is badly crumbled, especially on ends, with roughened sides.

The cement used passed all standard specifications.

The mortars were of three consistencies as measured by their flow on a flow table, the value of the flow for each consistency having been arbitrarily selected. Two types of specimens were made, 2 by 4-in. cylinders and 1 by 1 by 6-in. prisms. The prisms were filled from a 6 in. side and tamped with a steel rod, ½ in. in diameter. All mortar specimens were cured in water.

*Kinds of Tests:*

Three specimens of each kind as listed in Table V are being subjected to alternate freezing and thawing, the procedure being exactly the same as for the rock cylinders described earlier. The prisms are being kept with the same side down in the shallow trays in the freezer and will be so tested that this side will be under tension in the cross bending test. A similar duplicate set of specimens was stored in water to be broken at the same time as those subjected to the freezing-and-thawing tests. A third set was tested at 28 days.

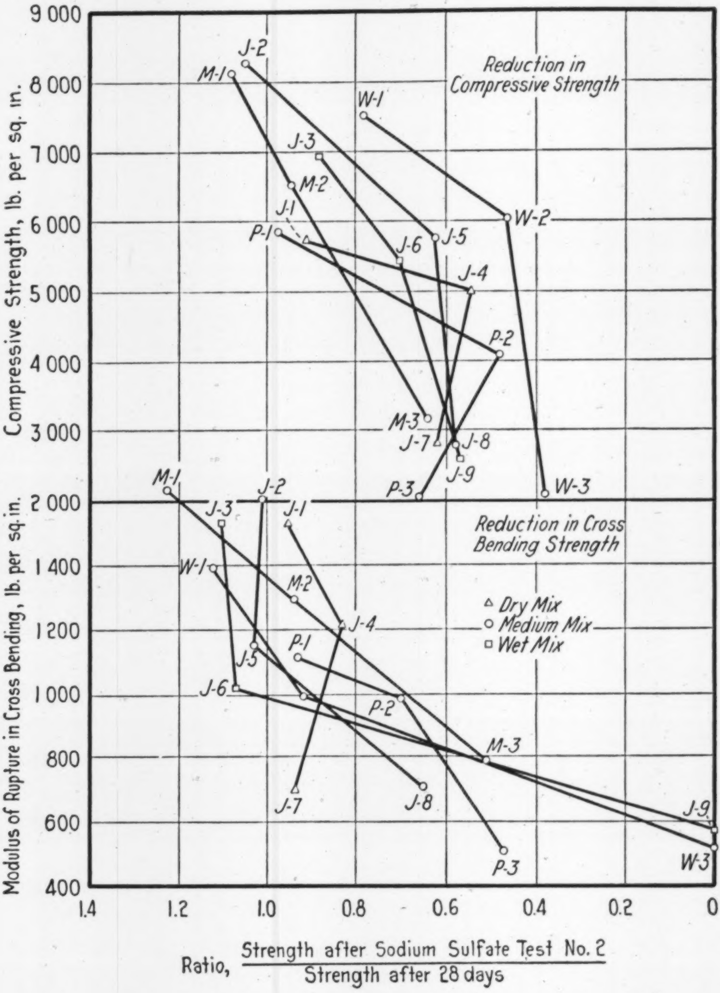

FIG. 2.—Reduction in Strength of Mortars After Sodium Sulfate Treatment.

A complete set of specimens was subjected to sodium sulfate test No. 2 and another to test No. 4. These tests were carried out as described earlier except that two-days immersion in water was allowed after the last drying in the oven.

Absorption tests were run on 8 cylinders of each mix, half being used to determine the effect of cold immersion and half the effect of boiling immersion. These tests were performed in the same way as the absorption tests on the rock.

### *Results of Tests:*

One complete set of mortar specimens, as outlined in Table V, is being subjected to alternate freezing and thawing. These have not been tested for strength as yet, but a visual examination was made after 80 reversals and the results are included in Table V.

Sodium sulfate tests Nos. 2 and 4 were also applied to complete sets of specimens. Test No. 4 was very erratic in its effect on the mortars and its value must remain undetermined until after the specimens being subjected to alternate freezing and thawing are tested for strength. Test No. 2 was quite destructive and gave some interesting results. An examination of Fig. 2 and the last column of Table V will show that the destructive effect of the test increased directly as the 28-day strength of the specimens decreased. Inasmuch as the 28-day strengths follow a water-cement ratio law, this destructive effect increases as the water-cement ratio increases. It would seem that something similar to test No. 2 might be developed into an extremely accelerated test for mortars, although account would have to be taken of the fact that a part of the destructive action might be due to chemical rather than physical properties of the salt.

In general, the estimated damage to the specimens by the alternate freezing and thawing appears to be in inverse order to their strengths. It is also of interest that the Wisconsin Michigan Power sand, which in every case gave the lowest 28-day strength for each mix, did not appear to weaken any faster under the sodium sulfate tests than the other mortars. The specimens subjected to the freezing and thawing seemed to be standing up rather better than some of the others.

There was no observable correlation between the results of the absorption tests and the weathering characteristics of the mortars.

In regard to the two forms of specimens, cylinders and prisms, it was found that the latter gave individual strength results that compared with the average very much better than did the cylinders. Weathering tests appeared to attack cylinders worse at the ends than at any other spot, making it difficult to satisfactorily cap them prior to determining their strength. This was avoided with the prisms in cross bending. The cross bending specimens also gave more uniform test results when tested from water at 28 days than did the cylinders.

## CONCLUSIONS

1. Based on a sample from each of 21 quarries representing four types of Wisconsin limestones, weathering tests consisting of 80 or more reversals of freezing and thawing of continually immersed samples occasioned relative disintegration comparable with what might have been expected from field inspection.

2. Accelerated weathering tests of five cycles—each cycle consisting essentially of immersion in concentrated sodium sulfate solution at 122° F. followed by immersion in concentrated sodium sulfate solution at 70° F. followed by drying at 212° F.—caused disintegration roughly comparable to that obtained by alternate freezing and thawing of the above limestones.

3. The above statements apply when the samples consist of crushed rock and when they consist of small rock cores.

4. Fineness moduli, based on mechanical analyses before and after weathering tests, are a fairly satisfactory measure for limestones of the disintegration caused by weathering tests provided the original sample is composed of fragments which are not of widely different sizes.

5. No relationship was found to exist between the weathering characteristics and the absorptive properties of the limestones listed above.

6. Based on tests of 18 mortar mixes representing four sands, an accelerated weathering test of five cycles—each cycle consisting essentially of immersion in concentrated sodium sulfate solution at 122° F. followed by immersion in concentrated sodium sulfate solution at 70° F. followed by drying at 212° F.—caused a weakening of the specimens inversely proportional to their 28-day strengths.

7. The accelerated weathering test mentioned in conclusion 6 is very severe, its destructive effect on mortars exceeding that of 80 reversals of freezing and thawing.

8. Basing judgment on the agreement between individual strength results and on the possibility of securing subsequent satisfactory strength tests, cross bending tests are the most satisfactory criteria of the destructive effects of weathering tests on mortars.

[For discussion of this paper, see page 835.—ED.]

# WEATHERING TEST PROCEDURES FOR CONCRETE

By F. H. Jackson[1]

In the following discussion, the term "masonry materials" will be understood to cover portland-cement concrete as ordinarily used in various types of monolithic construction as well as "precast masonry units," such as cast stone, concrete brick and tile, etc. It is realized that a strict interpretation of the term would limit the discussion to unit construction. However, the problem as applied to concrete in general is quite similar, although possibly not identical with the problems which are involved in the weathering of unit masonry. Moreover, most of the laboratory studies of weathering have been on concrete as used in general construction work and it is deemed advisable to include reference to this work in this discussion. It should be understood, also, that by "weathering resistance" is meant the ability of concrete to withstand the destructive effects of water, ice and heat as they normally occur, and does not include resistance to strong acid or alkali action, against which special precautions must be taken.

What are the factors which contribute to the maximum attainable resistance of concrete to the destructive forces which attack all exposed structures, including concrete, and how may the relative effects of these forces be investigated in the laboratory?

A laboratory test procedure which will simulate the effects of natural weathering may serve two very useful purposes. In the first place it may be used to study the fundamental principles involved in the design of concrete mixtures to resist weathering, thus making possible a determination of the relative influence of such variables as type and gradation of aggregates, quantity and quality of cement, quality of cement paste, method of curing, etc. This in turn makes it possible to show by actual demonstration in the laboratory how markedly the durability of concrete may be affected by lack of care in construction, or, in other words, by failure to apply these principles on the job.

It has been estimated that approximately 90 per cent of all concrete failures commonly ascribed to weathering are traceable to lack of care in construction rather than to any basic defect in either the materials used or in the theory of proportioning employed. This fact, in the author's opinion, cannot be emphasized too strongly or repeated too often. Until those in responsible charge of construction come to realize the important part which the construction operation itself plays in the production of weatherproof concrete, and provide for adequate inspection to insure it, any fundamental relationships which may be developed in the laboratory will only be of theoretical interest.

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[1] Senior Engineer of Tests, U. S. Bureau of Public Roads, Washington, D. C.

## LABORATORY TESTS

Although, as has been pointed out, by far the greater proportion of concrete failures by weathering are due to faulty construction methods, interest in the subject of weathering from the laboratory standpoint has so far been confined largely to studying the effect of character of aggregates on durability. Evidence in certain sections of the country that the use of unsound aggregates has been responsible for early weathering has stimulated a number of laboratory investigations along this line. These tests have so far been confined to efforts to reproduce in the laboratory the effects of frost action on concrete. In the case of the concrete itself, direct freezing and thawing tests have been made; in the case of aggregates, both freezing and thawing tests and soundness tests, using solutions of chemical salts such as sodium sulfate, have been employed. In both cases the development of crystal pressure within the pores of the concrete has been depended upon as the destructive agent. The significance of such a test lies in the fact that most of the weathering of concrete which is observed is probably due to either freezing of water within the pores of the concrete or to the deposition of crystals of chemical salts originally carried into the concrete in solution. In either event, failure is due to crystal pressure, so that a freezing-and-thawing test should from this standpoint be a measure of the resistance of the concrete to such action. An interesting discussion of crystal pressure with special reference to the effects of rapid temperature drop in reducing the solubility of chemical salts and thereby accelerating crystal growth is given by Anderegg (**1**)[1] in a paper presented at the 1929 meeting of the American Concrete Institute. A test procedure based on this action is suggested. Anderegg believes that "a large part of the damage in concrete that has been ascribed to freezing of water should be laid to salt-crystal pressure."

Other possible laboratory tests for durability, such as absorption and permeability, have been tried, the former largely in connection with studies of cast stone and concrete brick. Still other tests, such as alternate heating and cooling and alternate wetting and drying, have been suggested as of possible value, but have so far been tried only to a very limited extent.

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[1] The boldface numbers in parentheses refer to the reports and papers given in the list of references appended to this paper, see p. 794.

*Freezing-and-Thawing Tests:*

In a paper before the last annual meeting of the Highway Research Board(2) the author called attention to the desirability of investigating the technique of the freezing test. In practically every laboratory where investigations of the durability of concrete have been started, the necessity of obtaining early information as to the effect of type of aggregate on weathering resistance has forced the laboratory to adopt arbitrarily a test procedure without adequate preliminary investigation of the test method itself. The result has been the accumulation of considerable data based on the use of test methods which differ in many important details, some of which may affect markedly the rate of disintegration. For instance, the relative severity of action when specimens are immersed in water compared to the effect produced by freezing the saturated specimen in air is of interest. A suggested outline of procedure which might be used in standardizing this method was presented and discussed in the paper referred to above.

The author would be inclined to favor freezing the specimen either in air or partially immersed rather than totally immersed as being more nearly comparable with actual conditions. The recent suggestion of Mr. F. R. McMillan that freezing tests be made on samples of concrete which have been exposed to the weather under actual service conditions for a number of years should produce valuable information as to the significance of this test. As the matter now stands we really have very little information as to what the test actually means in terms of service behavior and can only draw broad comparisons between specimens subjected to the same test conditions. On this basis, interesting data have been obtained by Scholer(3) in Kansas, Goldbeck(4), National Crushed Stone Association, Gonnerman and Ward(5), Portland Cement Association and Lang and Hughes(6) in Minnesota.

The test procedures used by these investigators differ sufficiently to make comparisons difficult. It is significant to note, however, that in all cases the results which have been secured indicate in general that the strength of the mortar or concrete, as measured by the water-cement ratio used in the mix, is a more important factor in controlling the resistance to freezing and thawing than is the character of the aggregate employed, especially in the richer mixes. This is of course only true within certain broad limits, as it is well known that aggregates containing large quantities of obviously unsound materials, such as shale, etc., will produce unsound concrete regardless of the water-cement ratio used. Certain unpublished data obtained by the Bureau of Public Roads as the result of long-time freezing tests on concrete indicate that, as far as the usual types of coarse aggregate are concerned, the character of the material has very little effect on resistance to frost action. This conclusion, of course, applies only for the test condition used which in this case provided for freezing the saturated specimens in air at a temperature of about 4° F. Lower freezing temperature and freezing in water as employed by Scholer might have produced different relative effects. The question is, which method simulates more closely actual freezing conditions in the structure itself. A series of tests on concrete specimens taken from existing structures which have been exposed to natural weathering and subjecting these specimens to freezing tests conducted in various ways would throw much light on this problem.

*Heat Tests:*

The possibility that marked differences in the coefficients of expansion of the different constituents in hardened concrete may set up internal strains when the concrete is subjected to wide variations in temperature and thus produce an ultimate weakening of the structure has been advanced. A test procedure involving the alternate heating and cooling of concrete specimens containing aggregate of this character might yield interesting data. Valuable information might also be obtained by rapid alternations in moisture content produced by rapid drying of saturated specimens, tests to measure the rate of capillary flow of moisture and rate of absorption, etc. In this connection the reader is referred to a discussion by Hogentogler and Willis(7) in which the results of a number of interesting shrinkage tests on mortars are presented and the effect of controlling shrinkage on the durability of concrete is discussed.

*Sodium-Sulfate Tests:*

There is great interest and considerable difference of opinion among engineers at the present time regarding the significance of, as well as methods of conducting, the so-called "sodium-sulfate soundness test" for aggregates. The test procedure for coarse aggregate adopted a number of years ago by the American Association of State Highway Officials and described in U. S. Department of Agriculture

*Bulletin 1216* has been criticized by a number of investigators who point out that the test procedure is not defined with sufficient exactness to insure reproducible results. A suggested procedure which not only defines more exactly the test conditions but also sets up a quantitative method of measuring the effect of the test is described in the author's paper before the Highway Research Board referred to above(2).

The use of the sodium-sulfate test for determining the durability of fine aggregates is also of interest. Several methods of making the test have been suggested and here again the need for a standard procedure is very apparent. The method adopted by one of the state highway departments, which consists in making a separate determination on each size of sand rather than on a graded sample, appears to be based on sound reasoning. This method provides for testing each size separately and weighting the percentages of loss for each size in terms of the percentage of material of that size in the original sample. The sum of the weighted percentages of loss for each size is then reported as the percentage of loss for the entire sample. The method differs from the usual procedure in that the temperature of the solution is maintained at 77 to 82° F. instead of 70° F.

This method of determining soundness of fine aggregates is of special interest because it is being used for the first time, in so far as the author is aware, as an actual acceptance test.

The lack of any comprehensive data correlating the results of this test with service conditions makes its use as an acceptance test somewhat open to question, especially where the maximum percentage of loss is placed at a sufficiently low figure to reject material which does not contain a high percentage of known unsound constituents such as shale. The truth of the matter is that we know very little about the significance of the sodium-sulfate test as applied to coarse aggregates in spite of the fact that we have been using the test as an indication of possible unsoundness in stone for some years. We know still less about the test as applied to fine aggregates, and the question arises whether we are in a position at this time to adopt any definite percentage of loss in the sodium-sulfate test as a specification requirement which will reject materials in cases where a mineral examination fails to reveal a high percentage of unsound materials. For an interesting discussion of the sodium-sulfate test for coarse aggregate, the reader is referred to the Symposium on Aggregates held by the American Society for Testing Materials in 1929 and reported in the *Proceedings* of that year(8).

## REFERENCES

(1) F. O. Anderegg, "The Mechanism of the Corrosion of Portland-Cement Concrete with Special Reference to Crystal Pressure," *Proceedings*, Am. Concrete Inst., Vol. 25, p. 332 (1929).

(2) F. H. Jackson, "Relation Between Durability of Concrete and Durability of Aggregates," *Crushed Stone Journal*, Vol. 6, December, 1930, p. 5.

(3) C. H. Scholer, "Durability of Concrete," *Proceedings*, Highway Research Board (1930).

(4) A. T. Goldbeck, "Some Observations on Accelerated (Sodium-Sulfate) Soundness Tests," *Pit and Quarry*, Vol. 19, pp. 47, 62 (1929); also, *Crushed Stone Journal*, Vol. 5, July, 1929, p. 13. Abstracted in *Chemical Abstracts*, Vol. 24, p. 1195 (1930).

(5) H. F. Gonnerman and G. W. Ward, "Durability Studies of Aggregate and Concrete," Association of State Highway Officials of the North Atlantic States, p. 45 (1931).

(6) F. C. Lang and C. A. Hughes, "Discussion of Papers on Relation Between Durability of Concrete and Durability of Aggregates," *Proceedings*, Highway Research Board, p. 116 (1930).

(7) C. A. Hogentogler and E. A. Willis, "The Phenomena of Shrinkage and Its Effects upon the Integrity and Durability of Concrete Pavements," *Proceedings*, Highway Research Board, p. 164 (1930).

(8) Symposium on Mineral Aggregates, *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part II, p. 740 (1929).

[For discussion of this paper, see page 835.—ED.]

# WEATHERING TEST PROCEDURES FOR CLAY PRODUCTS

By H. D. Foster[1]

Weathering specifically denotes a change in form, shape or condition (usually deterioration) of materials subjected to the atmosphere and atmospheric changes. A study of weathering should involve not only a study of the action of freezing water within the pores of the material but also the chemical action of acids from the air, the erosion or abrasion of sharp wind-blown particles, the expansion or contraction due to climatic changes in temperature, etc. Well-fired clay products are highly resistant to the attack of acids that are normally present in the atmosphere. Moreover, these products do not shrink and swell with changes in moisture content more than a negligible fraction of the amounts shown by concrete and some natural stones. Hence the commonly accepted view seems to be correct, that the chief agents of weathering causing deterioration of clay products are frost action and the crystallization of salts in the pores.

The conditions of service should govern the type of weathering test applicable to materials. Clay products in service are used under three weathering conditions, namely, underground as in drain tile, at the ground level as in foundations, and in wall surfaces which are not in contact with the soil. Weathering tests have been made by the duplication of actual weathering conditions such as fully or partially saturating the specimens with water and then freezing to obtain the disruptive effect of the formation of ice in the pores of the product, or by simulated freezing with the formation of sodium-sulfate crystals and their consequent expansion in the pores of the ware.

Weathering requirements have been set down in various specifications of the Society, but for the most part these requirements have been withdrawn or have rarely been used. There are two specifications relating to masonry materials, the Standard Specifications for Drain Tile (C 4 – 24) and for Hollow Tile (C 34 – 30) which still carry weathering or freezing requirements. In both of these specifications it is required that, prior to freezing, the specimens be saturated by immersion in water at room temperatures (65 to 75° F.) for 72 hours. Specimens of drain tile are immersed in at least ½ in. of water during the freezing and the freezing is continued until all of the water is frozen. They are then thawed by immersion in water at room temperature. The specifications for structural clay tile do not specify a definite test procedure but only state that the time and temperature of freezing and thawing be such as to insure full freezing and thawing throughout the specimen. Nor do these specifications state whether any part of the specimen should be immersed in water during the freezing. It is generally supposed that the specimens be not in contact with water during the freezing.

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Drain tile are acceptable, so far as the weathering requirements of these specifications are concerned, if they will pass 24 cycles of freezing and thawing for farm drain tile, 36 for standard drain tile and 48 for extra-quality drain tile. Structural clay tile are required to withstand 100 alternate freezings and thawings to be acceptable for use in exterior walls. The difference in requirements for these two materials is probably due to differences in the severity of the methods of test. The drain tile are in contact with the water during the freezing which would tend to cause an increase in their moisture content during freezing, whereas structural clay tile, being frozen in air, would not gain moisture during freezing.

A study of the different methods of making absorption tests has shown that immersion in water at room temperature for three days will give from 60 to 80 per cent complete saturation. If the initial saturation were obtained by boiling and then cooling in water to room temperature, the percentage saturation would be 85 to 95 per cent complete. The percentage saturation will influence the results of the freezing tests. Schlick found in connection with his tests that tile when boiled in order to obtain saturation prior to the freezing of the specimens gave 18 per cent greater loss for 50 cycles of freezing and thawing than was obtained for those specimens which were saturated by immersion in water at room temperatures for 36 or 72 hours. Similar increases were obtained when the pieces were boiled during the thawing process. These increases were additive when the specimens were boiled both in obtaining initial saturation and in thawing. This increased loss was more perceptible for pieces fired to a medium degree of maturity than for the softer fired pieces. Kreuger has shown that the loss in weight due to freezing is practically nothing when the piece is less than 70 per cent saturated, and immaterial when the piece is from 70 to 80 per cent saturated, but increases rapidly when 80 per cent saturation is exceeded.

Wall surfaces may be wetted thoroughly, but before freezing takes place or at least before the interior of the wall is cooled, so much of the water in the pores has escaped that little damage can be done.

A hollow wall, even if completely saturated by rain, will probably never retain all of its moisture until freezing occurs. The percentage saturation which it would retain would depend upon its individual characteristics, the materials having large and open pores draining readily and those having small pores retaining their water for a long time. Inasmuch as it is impossible to estimate the completeness of the saturation at the time freezing takes place, wall materials should probably be subjected to such tests as will more nearly duplicate the conditions of use. This can be done only by the use of large units or of small panels of small units which can be saturated and then subjected only on the one side to freezing temperatures. As such a test cannot readily be made a laboratory test, the nearest approach to service conditions would be to use small pieces which have been partially saturated and then permitted to drain a small amount before freezing. They should not be in contact with water during freezing.

The freezing test is burdensome and requires an unduly long time even when carried only far enough to give an indication of early disintegration of the pieces. A shorter test has been tried. This shorter test consists of causing the formation of crystals of sodium sulfate in the pores of the specimen. Edward Orton, Jr.,[1,2] carried out a series of tests of brick in which he subjected samples to both the sodium-sulfate treatment and the alternate freezing and thawing treatments. He controlled the temperature of the formation of the crystals by using an oven at a temperature of between 230 and 250° F., so that the anhydrous sodium-sulfate crystal would form rather than the decahydrate which forms at room temperature. The decahydrate crystal has a much larger volume and consequently would have more severe disruptive effect on the piece. For this reason the sodium-sulfate treatment should be used under controlled conditions. It was found from this series of tests that four of the sodium-sulfate treatments was equal to twenty-five of the freezing and thawing treatments when the freezing was done with the specimen in contact with water.

The effect of the sodium-sulfate treatment can be evaluated in terms of the freezing-and-thawing treatment when the specimens are in contact with water, but neither can be evaluated with the results obtainable from materials under conditions like those most common in wall surfaces.

From investigations and examinations made by the Building

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[1] Edward Orton, Jr., "A Comparison Between the Absorption, Crushing Strength and Resistance to Artificial Freezing of Some Ohio Building Bricks," *Transactions*, Am. Ceramic Soc., Vol. 18, p. 686 (1916).

[2] Edward Orton, Jr., "A Study of the Proposed A.S.T.M. Tentative Specifications for Building Brick and Correlation of Their Requirements with Sodium-Sulfate Treatment and Actual Freezing," *Proceedings*, Am. Soc. Testing Mats., Vol. XIX, Part I, p. 268 (1919).

Research Board[1, 2] in England, it appears that the disintegration of clay products in that country is due largely to the disruptive effects of the crystallization of salts within the pores. Probably a similar statement would apply to some parts of this country, especially in warm humid climates. The Board suggests that although the results of crystallization tests may not be a reliable index of the resistance of materials to frost action, some form of this test may serve as a measure of their resistance to this type of action in service.

Glazed wares, such as glazed terra cotta and brick, sometimes exhibit a crazing of the glaze that takes place after the wares have been in service. H. G. Schurecht has shown[3] that this delayed crazing of glazes may be caused by the slow expansions of the body during the exposure of the ware, the expansion of the body usually being greater than that of the glaze. Laboratory tests indicate that the expansion is caused by the absorption of moisture, as the expansion is not removed by heating to a temperature somewhat greater than 210° F. but may be removed by heating to about 480° F. Most clay ware is not subject to this type of expansion, but those that are cannot be detected from the results of strength and absorption tests. It has been found, however, that a close correlation exists between the expansions produced by steaming in an autoclave at 150 lb. per sq. in. pressure for one hour and the tendency to expand in service.

From the above it can be concluded that if the materials are to be used in connection with the ground or elsewhere adjacent to an available supply of water, test specimens should be frozen partially immersed or they can be subjected to the sodium-sulfate treatment if care is taken to insure the formation of the anhydrous crystal rather than the decahydrate. If the materials are to be used in wall surfaces the only test directly applicable would be a service test, but since this is impractical the nearest approach to the service test would be obtained when the specimens are saturated and partially drained before being frozen. Inasmuch as it always takes some time to lower the temperature of the freezing chamber after the insertion of the specimens, this draining can take place during the cooling stage provided the specimens are placed in wire baskets.

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[1] W. A. McIntyre, "Factors Governing the Durability of Clay Building Materials," *British Clayworker*, Vol. 37, p. 195 (1928).

[2] W. A. McIntyre, "Investigations into the Durability of Architectural Terra Cotta and Faience," *Special Report No. 12*, Department of Science and Industrial Research, Building Research Board of Great Britain (1929).

[3] H. G. Schurecht, "Methods for Testing Crazing of Glazes Caused by Increase in Size of Ceramic Bodies," *Journal*, Am. Ceramic Soc., Vol. II, p. 271 (1928).

# WEATHERING TEST PROCEDURES FOR STONE[1]

By D. W. Kessler[2]

## Freezing Tests

In this country frost has been proclaimed the chief weathering agent and hence freezing tests have occupied the attention of the laboratory. A study of the technical writings will show a considerable diversity of test procedures aimed at the determination of frost resistance. As far back as 1856 we find experimental data on this property. The interesting conclusion of one experimenter was that it would require 10,000 years to weather one inch from the exposed face of marble. This was based on 50 cycles of freezing and thawing.

A determination of frost resistance is a slow and laborious process, so the usual procedure has been to limit the number of cycles and attempt to measure the injury to the specimens by means of strength or weight losses. The work of Buckley**(1)**[3] for the state geological departments of Wisconsin and Missouri has provided considerable pioneer information as to this method. His procedure consisted of placing the saturated specimens outdoors during winter nights to freeze and soaking them during the day. After 35 cycles most of the materials showed measurable weight losses but a rather large percentage indicated higher strengths after freezing. Similar results from this method have been recorded by Parks**(2)** for the "Building and Ornamental Stones of Canada" and by Kessler**(3)** for the Commercial Marbles of the United States. Small weight losses cannot always be attributed to frost action and the strength-loss criterion is less satisfactory for two reasons: first, the strength losses due to 30 or 40 freezings are so small for most building stones that they may be much less than the errors of strength measurements; and, second, the variation in strength from one specimen to another is often found to be 10 per cent or more, and as the original strength of the specimens being frozen must be assumed to be equal to that obtained on other specimens, the predetermined original strength may be in error by a considerable amount.

Such considerations have led to attempts to accelerate the action by more severe conditions such as freezing in water, boiling before freezing or saturating the specimens under reduced air pressures. Whether such methods give results of any particular value is questionable. A recent test on a limestone that endured for several centuries in France showed that the material could be completely disintegrated by 18 cycles when the specimens were frozen while partly immersed in water. The same procedure indicates low frost resistance for some of our domestic limestones which have shown good weathering qualities in service. However, it appears that for many building stones there is a better correlation between results obtained by freezing in water and the weathering in service than in the cases cited above.

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[1] Publication approved by the Director of the Bureau of Standards of the U. S. Department of Commerce.

[2] Civil Engineer, U. S. Bureau of Standards, Washington, D. C.

[3] The boldface numbers in parentheses refer to the reports and papers given in the list of references appended to this paper, see p. 802.

The uncertainties involved in measuring small strength and weight losses resulting from a small number of cycles have led some experimenters to discard this method and instead to continue the cycles until a noticeable amount of decay occurs and when a certain stage is reached the number of cycles is taken as the measure of frost resistance. The Building Research Board(4) has used this procedure with very rapid freezing as follows: The specimens after saturation by gradual immersion, are wrapped in muslin to collect any detached fragments and immersed 24 hours in the cold brine at about −14° C. It is stated that this procedure produces as much decay in one or two freezings as a large number of freezings where the specimens are frozen while partly immersed in water. A less severe but a very tedious test is reported in Bureau of Standards *Technologic Paper No. 349*(5) in which specimens were immersed in water for 14 days, then placed in wire baskets and stored in a freezing chamber at about −14° C. until frozen. The specimens were thawed by immersion in water at room temperatures for 30 minutes. From two to four cycles were completed in a day and the process was continued until at least one-tenth of the specimen had crumbled away. The number of cycles was the criterion. The water absorbed by the material increased as the process continued until from 70 to 90 per cent of the pore space was filled.

A German procedure for frost resistance requires that the saturated specimens be placed in a freezing chamber, the temperature of which can be lowered to −15° C. in four hours and held two hours more. After each freezing the specimen is immersed in its own individual water bath in order to collect the loosened fragments. Before each freezing, the specimens are weighed and after 25 cycles the loosened particles are determined by evaporating the thawing water. Decision is based on the weights of the saturated specimens after freezing, the dry weights of the loosened particles and the final appearance.

## SALT CRYSTALLIZATION TESTS

Crystallization tests were originated with the idea that they produce an action similar to that of frost. Several experimenters have attempted to prove a relationship between the sodium sulfate test and the freezing test, but in general it must be admitted that no relation has been established. Luquer(6) experimented with various types of stone, comparing the results of the sodium sulfate tests with freezing-and-thawing on similar specimens. There was no definite indication of a relation. Hatfield(7) experimented along similar lines and although his results are somewhat more encouraging he did not attempt to establish a relation.

Sodium sulfate in solutions varying in strength from 14 per cent to saturation at room temperatures have been used. Drying conditions have also been varied considerably. Luquer, after soaking the specimens for 12 hours in the solution, hung them up in a dark room to dry for 12 hours. Some have steamed the specimens before drying while others have dried them in an oven with damp sand to maintain damp conditions during the first stages. Evidently the most severe conditions are attained when crystals of $Na_2SO_4 \cdot 10\ H_2O$ are formed. This form does not ordinarily exist at temperatures above 33° C. It would seem from tests of this kind made by the author that the strongest disrupting action occurs while the specimens are in the solution rather than while they are being dried, as they usually disrupt in the solution.

The Building Research Board(4) has experimented with magnesium sulfate, calcium sulfate, potassium sulfate, potassium nitrate, and sodium thiosulfate solutions as disintegrating agents. That institution expresses the belief that crystallization tests produce a type of decay very similar to that occurring in buildings. In England the weathering agents seem to be mainly confined to crystallizing salts, particularly calcium and magnesium sulfates. In Bureau of Standards *Technologic Paper No. 349*(5) is described a procedure in which a 14-per-cent solution of sodium chloride was used. This salt seems to give somewhat less severe action than sodium sulfate, and there appears to be no definite correlation of the results with frost action. On some materials it proved to be more severe than frost action while for the other material the relation was reversed. In Bureau of Standards *Technologic Paper No. 350* is a description of crystallization tests with sodium carbonate and sodium bicarbonate on marble. On this material these salts seem to have a much more severe effect than sodium sulfate.

## MISCELLANEOUS TESTS

Various investigators have experimented with the solvent action on stone of carbonated waters and corrosive atmospheric gases. Although the action of atmospheric acids appears to be of considerable importance in relation to some types of stone, no definite test procedures have been proposed. A three-stage test, which consists of three hours soaking in a 1-per-cent solution of HCl and $H_2SO_4$, 18 hours in the freezing chamber and 3 hours in the drying oven, is being studied at the Bureau of Standards. Some materials which are extremely resistant to frost action alone are found to have a rather low resistance in this test. Furthermore, the failure of some of the materials is much like that of the materials in actual weathering.

Recently it became necessary to devise a weathering test for slate. It was found that this material showed no appreciable disintegration in the freezing test even after three or four thousand cycles. An examination of numerous samples of weathered slate showed that the decayed portions contained a considerable amount of calcium sulfate, although the fresh slate contained practically no sulfates but calcite in considerable amounts. It was found that the calcite could be converted to calcium sulfate by two means: first, by soaking the specimens in a dilute solution of $H_2SO_4$ or, second, by several cycles of soaking in water and drying. Either process produces decay which is evidently caused by the pressure of the gypsum crystals, and the type of decay resembles that which occurs in actual weathering. The procedure now being used consists of seven days immersion in a 1-per-cent solution of $H_2SO_4$ and a determination of the depth of decay.

In this discussion it has been the purpose to point out some of the difficulties that may be encountered in trying to estimate the durability of a material from the results of a weathering test. Before a wise choice of a weathering test can be made for a given material, one should know the major weathering agents for that type of material. Some thirty years ago Julien (8) advocated a study of the properties of the various stone-forming minerals. To this might also be added that it is very desirable to determine what chemical changes take place when various combinations of minerals are exposed to various weathering processes.

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(3) D. W. Kessler, "Physical and Chemical Tests on the Commercial Marbles of the United States," U. S. Bureau of Standards *Technologic Paper No. 123* (1919).

(4) 1927 Report of the Building Research Board, Department of Scientific and Industrial Research, Westminster, England.

(5) D. W. Kessler and W. H. Sligh, "Physical Properties of the Principal Commercial Limestones used for Building Construction in the United States," U. S. Bureau of Standards *Technologic Paper No. 349* (1927).

(6) L. M. Luquer, "The Relative Effects of Frost and Sulfate of Soda Efflorescence Test on Building Stones," *Transactions*, Am. Soc. Civil Engrs., Vol. XXXIII. March, 1895.

(7) R. G. Hatfield, "Experimental Tests of Building Stones," *Transactions*, Am. Soc. Civil Engrs., Vol. XLVIII, p. 145 (1873).

(8) Alexis Julien, "Building Stones," *Journal*, Franklin Inst., April, May and June, 1899.

[For discussion of this paper, see page 835.—ED.]

# BIBLIOGRAPHY ON WEATHERING OF NATURAL STONE

By D. W. Kessler[1]

## 1851 to 1899

(1) W. R. Johnson, "Experiments on American and Foreign Stones to Determine Their Strength," *American Journal of Science*, Vol. XI, Series 2, January, 1851.

An evaluation of numerous stones based on compressive strength.

(2) Joseph Henry, "On the Mode of Testing Building Materials and an Account of the Marble Used in the Extension of the U. S. Capitol," *American Journal of Science*, Vol. 72, p. 30 (1856).

Specimens of marble frozen 50 times and amount of material lost was computed to be equal to a depth of 0.0001 in.

(3) D. T. Ansted, "On the Decay and Preservation of Building Materials," *Journal*, Franklin Inst., September and October, 1860.

A discussion of weathering effects on various types of stone based on observations of structures in Great Britain.

(4) Edward Hull, "Building and Ornamental Stones," MacMillan and Co., New York City (1872).

One chapter on selection of stone with regard to climate and the nature of the atmosphere.

(5) R. G. Hatfield, "Experimental Tests of Building Stones," *Transactions*, Am. Soc. Civil Engrs., Vol. XLVIII, p. 145 (1873).

Results of freezing tests and sodium sulfate tests on a few samples of marble, limestone, sandstone, serpentine and coquina.

(6) C. P. Townsly, "The Decay of Building Stones," *Van Nostrand's Engineering Magazine*, Vol. XIV, p. 535 (1876).

Brard's test on 32 samples of stone (14 cycles) with weight loss determinations. Materials were granite, marble, limestone, sandstone and gneiss.

(7) Alexis Julien, "The Durability of Building Stones in New York City and Vicinity," Tenth Census of the United States, Vol. 10 (1880).

Discusses various weathering agents and their effects as noted on buildings. From an extended study of buildings he estimates the life of various types of stone in that vicinity as ranging between 5 years and many centuries.

(8) A. A. Julien, "The Decay of Building Stones of New York," *Transactions*, New York Academy of Science, Vol. 2, pp. 67, 120, 138 (1882 to 1883).

(9) Board of Capitol Managers, Report of the Committee on Building Stones to the Board, Denver, Colo., July 3, 1884.

Freezing tests on 31 samples of stone including sandstone, limestone, marble, dolomite and lava. Apparently only two cycles of freezing and thawing were made.

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[1] Civil Engineer, U. S. Bureau of Standards, Washington, D. C.

(10) "The Resistance of Building Materials to Frost," ***Van Nostrand's Engineering Magazine***, Vol. 34, p. 44 (1886).

Results of freezing tests on 10 samples of stone, 2 to 24 cycles giving weight losses varying from 0.08 to 5 grams.

(11) Thomas Egleston, "The Cause and Prevention of Decay in Building Stones," *Transactions*, Am. Soc. Civil Engrs., Vol. XV, p. 647 (1886).

Discussion of weathering based on studies of several buildings in New York City. Stress is placed on chemical weathering and abrasive action of wind-blown sand.

(12) J. C. Smock, "Building Stone in New York," *Bulletin*, New York State Museum, Vol. 2, No. 10 (1890).

Chapter on durability and causes of decay.

(13) Hudson Beare, "Building Stones of Great Britain," *Minutes of Proceedings*, Inst. Civil Engrs., Vol. 107, p. 341 (1891).

Freezing tests on a few limestones, sandstones and granites. Only slight effects noted due to the limited number of cycles.

(14) "The Selection of Building Stones," ***Engineering News***, March 30, 1893, p. 305.

Recommendations based on the report of C. H. Porter to the commission charged with the erection of the New York State Capitol. Stone should be hard, fine grained, uniform texture and nearly impervious. Stone in a ledge covered by lichens is apt to be durable.

(15) L. M. Luquer, "The Relative Effects of Frost and Sulfate of Soda Efflorescence Test on Building Stones," *Transactions*, Am. Soc. Civil Engrs., Vol. XXXIII, March, 1895.

Comparative tests on 12 samples of stone including limestone, dolomite, granite, gneiss, norite and sandstone show no definite correlation.

(16) H. F. Bain, "Properties and Tests of Iowa Building Stones," *Iowa Geological Survey*, Vol. 8 (1898).

Freezing tests, 16 cycles, on 15 samples of limestone produced very slight weight losses and no appreciable loss of strength.

(17) E. R. Buckley, "Building and Ornamental Stones of Wisconsin," *Bulletin IV*, Wisconsin Geological Survey (1898).

An elaborate description of the stones of Wisconsin and discussion of weathering agents. Freezing tests, sulfurous acid gas and carbonic acid gas tests on numerous materials reported.

(18) A. Marston, "Experiments on Stone," *Proceedings*, Iowa Engineering Soc., January, 1898, p. 123.

Freezing tests on 15 stones including limestone, sandstone and marble. Strength and weight losses determined after 20 freezings.

(19) G. P. Merrill and E. B. Mathews, "The Building and Decorative Stones of Maryland," *Maryland Geological Survey*, Vol. 2, p. 2 (1898).

Thermal expansion and contraction cause permanent disarrangement of the component parts, opening up rifts which absorb water when frost becomes a considerable factor. Thermal effects are the most important cause of decay in granites and gneisses. Serpentines weather unevenly due to presence of calcite. Limestones weather by solution while the weathering of the sandstone depends on the nature of the binding material.

(20) E. R. Buckley, "Color and Decay of Stone," *Stone*, October, 1899, p. 426.

Article stresses the injurious effects on stone due to certain methods of quarrying and setting.

(21) Alexis Julien, "Building Stones," *Journal*, Franklin Inst., April, May and June, 1899, pp. 257, 378, 430, respectively.

Different types of stone weather from different causes, hence no single weathering test is applicable to all types. Discusses effects of internal lubricants: water, petroleum, bitumen, mica plates, graphite. Advocates an extensive study of the properties of the various stone-forming minerals.

1900 to 1919

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Freezing tests and sulfurous acid tests on limestone and sandstone.

(23) F. W. Hoyt, "Choice of Building Stones," *American Architect*, August, 1906.

Life of various kinds.

(24) J. Malette, "Note on the Resistance of Building Stone to Frost," *Builder* (London), September 15, 1906.

(25) T. L. Watson, "Building and Ornamental Stones of North Carolina," *Bulletin No. 2*, North Carolina Geological Survey (1906).

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(26) R. D. George, "The Characteristics of Good Building Stones," *Ores and Metals*, March 20, 1907, p. 123.

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Pressure tests on limestone, sandstone and granite (*a*) dry (*b*) saturated with water and (*c*) frozen 25 times.

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A text book of 551 pages, 40 pages devoted to weathering action and selection of stone.

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(67) "Some Aspects of Weathering of Building Materials," *Builder* (London), Vol. 135, p. 4477 (1928).

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Decay in cities is attributed to acids from flue gases. Recommends waterglass and $CaCl_2$ treatment.

## 1929 to 1931

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(82) "Efflorescence on Hard Facing Stones Around Cement Joints," *Ciment*, Union Technique du Batiment et des Travaux Publics, Vol. 34, No. 5, p. 245 (1929).

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(83) "The Decay of Stone in Buildings and Monuments," *Quarry*, Vol. 34, No. 383, p. 8 (1929).

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An able discussion of physical properties affecting weathering resistance. Experiments on solubility in water containing varying amounts of $CO_2$.

(92) A. N. Murray and W. W. Love, "Action of Organic Acids upon Limestone," *Bulletin*, Am. Assn. Petroleum Geologists, Vol. 13, p. 1467 (1929).

Water analyses from 8 places showed organic acid contents varying from 1.8 to 34.9 p.p.m. Organic acids divided into two groups (*a*) those which are formed in the plant (*b*) those produced by bacterial decomposition.

(93) "Some Little Known Causes of Stone Decay," *Journal*, Soc. Chemical Industry, Vol. 49, No. 16, p. 324 (1930).

Air borne sea salts and magnesium sulfate formed from treatment of silicofloride of magnesium or waxy preservatives improperly applied may lead to trouble.

(94) "A Study of Weather Action on Marble," U. S. Bureau of Standards *Technical News Bulletin No. 161*, September, 1930.

Strength and absorption measurements on a block of marble exposed in a building 70 years old showed strength losses varying from 60 to 70 per cent. Absorption measurements indicated that the porosity had increased nearly 300 per cent near the exposed faces and about 40 per cent at the center of the block.

(95) D. W. Kessler, "Tests for Weathering Characteristics," *Rock Products*, Vol. 33, No. 10 (1930).

Some materials well adapted to continuously damp conditions but do not survive where alternately wet and dried. Other materials decay from alternating conditions of moisture and dryness without exposure to frost or acid conditions.

(96) "Destructive Weathering of Slate," U. S. Bureau of Standards *Technical News Bulletin No. 164*, December, 1930, p. 121.

Slates high in calcitic impurities weather from a physico-chemical process involving first a chemical interchange between the calcite and pyrite with the conversion of calcite to gypsum, the necessary oxygen being drawn from the air. The internal stresses cause disruption of the slate.

(97) A. M. Giles, "The Use and Abuse of Building Materials," *Builder* (London), Vol. 138, p. 4535 (1930).

Weathering and preservation of granite, sandstone and limestone discussed. Method of preservation mentioned.

(98) L. Maric, "The Weathering of Granites," *Chemical Abstracts*, Vol. 24, No. 11, p. 2697 (1930).

Composition found not to change materially. Ferric oxide, potassium oxide and water increased. Sodium oxide and lime from oligoclase decreased slightly, silica and aluminum almost unchanged.

(99) W. R. Warnes and D. Northall-Laurie, "Sea Salt as Cause of Decay in Limestone," *Architect's Journal*, Vol. 71, No. 1835, p. 466 (1930).

A discussion of NaCl carried into stone through the spray or by direct exposure of stone to sea water before use.

(100) D. W. Kessler, "Weathering of Stone," *American Architect*, February, 1931, p. 28.

A discussion of weathering effects on various types of stone masonry, giving results of tests on marble from a building 70 years old.

# BIBLIOGRAPHY ON THE WEATHERING OF CONCRETE

BY F. R. MCMILLAN[1]

## A. FREEZING AND THAWING AND OTHER ACCELERATED SOUNDNESS TESTS ON AGGREGATES, BUILDING STONE, CONCRETE, ETC.

(101) L. J. Vicat, "Treatise on Calcareous Mortars and Cements," translated from French by J. T. Smith, J. Weale, London (1837).

Following is from Appendix to Vicat's book by Smith: "Whilst we were engaged with these researches, Mr. Brard (then director of the collieries of Lardin, in the department of Dordogne), tried to distinguish the stones which are injured by frost, from those which are not so, by substituting for the expansive force of congealing water, that of an easily crystallizable salt, the sulfate of soda. No sooner were we made acquainted with this happy idea, then were eager to try it upon our mortars. In conformity with the directions of the able mineralogist whom we have just named, a hundred specimens were impregnated with a warm, saturated solution of the sulfate of soda, and then exposed in a loft to the temperature of 25 to 30° C. (77 to 86° F.), and lastly, washed from day to day with pure water. These specimens were not long in giving signs of degradation."

(102) M. O. Withey, "Soundness Tests for Coarse Aggregates," *Proceedings*, Fourth Annual Meeting Highway Research Board, p. 108 (1925).

Four types of crystallization tests used for determining soundness of stone and clay products: (*a*) freezing and thawing test; (*b*) sodium sulfate test; (*c*) sodium chloride test; and (*d*) alkali test. Method of making tests and results of soundness tests on various aggregates.

(103) "Freezing and Thawing Tests of Concrete," *Highway Research News*, Vol. 1, p. 4. November 15 to December 1, 1925.

U. S. Bureau of Public Roads sodium sulfate tests on concrete and aggregate.

(104) "Durability Tests of Aggregates," U. S. Bureau of Standards *Technical News Bulletin*, July, 1927, p. 7; also, *Rock Products*, Vol. 30, p. 51, September 17, 1927.

Alkali, sodium sulfate, sodium chloride, and boiling and drying tests for soundness of aggregates.

(105) "Conditions Affecting Durability of Concrete," Report of Subcommittee X of Committee C-9 on Concrete and Concrete Aggregates, *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 349 (1927). Abstracted in *Building Science Abstracts*, Vol. 1, p. 137, May, 1928.

Sodium sulfate soundness test applied to limestones appears to be reliable.

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[1] Director of Research, Portland Cement Assn., Chicago, Ill.

(106) C. H. Scholer, "Some Accelerated Freezing and Thawing Tests on Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 472 (1928); also, *Engineering and Contracting*, Vol. 67, p. 455, September, 1928.

Method for testing durability of concrete by alternate freezing and thawing at low temperature; data are included covering field investigation of disintegrating concrete structures which show relation between field conditions and results of laboratory investigation by accelerated freezing tests; data include results on unsound aggregates, varying values of water-cement ratio, and variations in rate of freezing under different conditions of exposure.

(107) "Influence of Durability Test on Strength of Concrete Aggregate," U. S. Bureau of Standards *Technical News Bulletin*, No. 134, p. 87 (1928).

Tests on cylinders about 2 in. in diameter and 2 in. high, and on 2-in. cubes cut from limestone block selected at random. The samples were boiled in water for 6 hours and then dried at 110° C. for 17 hours, this cycle being repeated 100 times. Comparative compressive strength tests carried out with untreated samples cut from the same blocks showed an average reduction of 38 per cent in the strength as a result of the cycles of boiling and drying.

(108) W. Petry, "Deterioration of Concrete," *Tonindustrie Zeitung*, Vol. 52, p. 398 (1928). Abstracted in *Building Science Abstracts*, Vol. 1, p. 168, June, 1928.

(109) G. F. Loughlin, "Usefulness of Petrology in Selection of Limestone," *Rock Products*, Vol. 31, p. 50 (1928).

Tests comprised among others weight, absorption, slaking or sodium sulfate tests and freezing and thawing.

(110) "Devices that Aid Laboratory Studies in Concrete," *Engineering News-Record*, Vol. 100, p. 432 (1928).

Tests of repeated freezing and thawing at University of Colorado testing laboratory.

(111) W. H. Batchelder, "Significant Tests of Frost Action on Concrete," *Engineering News-Record*, Vol. 101, p. 882, December 13, 1928. Abstracted in *Building Science Abstracts*, Vol. 2, p. 73, February, 1929.

Series of concrete beams (6 by 6 by 30 in., 1:2:3 mix, natural gravel and trap rock) tested (1) cast at 70° F. and at 40° F., exposed to (1) freezing for 14 days, (2) for 16 hours, (3) freezing cycles of 16 hours freezing and 8 hours thawing, (4) 5 freezing cycles, and (5) curing for 28 days.

(112) R. Grün, "Investigations of Concrete in Freezing Chambers," *Zement*, Vol. 17, pp. 1371, 1400, 1471 (1928).

(113) C. A. Wiepking, "Notes on Freezing (Concrete) Block," *Rock Products*, Vol. 31, p. 65 (1928). Abstracted in *Building Science Abstracts* (London), Vol. 1, pp. 107, April, 1928.

Results of effect exhibited by gravel concrete building block after 100 alternate freezings and thawings.

(114) A. T. Goldbeck, "Some Observations on Accelerated (Sodium Sulfate) Soundness Tests," *Pit and Quarry*, Vol. 19, No. 3, pp. 47, 54, 62 (1929); also *Crushed Stone Journal*, Vol. 5, p. 13, July, 1929. Abstracted in *Chemical Abstracts*, Vol. 24, p. 1195 (1930).

Comparison between sodium sulfate and freezing-and-thawing tests.

(115) G. W. Burke, "Relation of Chemical Test Methods and Sulfate Resistance of Different Brands of Portland Cement," *Rock Products*, Vol. 32, p. 79, June 8, 1929.

Suggests that sulfate resistance of portland cements might best be measured by treating test specimens with 10 to 15 per cent solution of sodium sulfate.

(116) "Resistance to Freezing and Weathering of Concrete," *Die Portlandzemente*, April, 1929, p. 32.

(117) H. F. Kriege, "Stability of Chert," *Rock Products*, Vol. 32, p. 50, April 27, 1929.

Freezing and thawing and sodium sulfate tests on cherty aggregate and concrete made from it.

(118) F. H. Jackson, "Relation Between Durability of Concrete and Durability of Aggregates," *Crushed Stone Journal*, Vol. 6, p. 5, December, 1930.

Thermal expansion, sodium sulfate test and interpretation.

(119) F. R. McMillan, "Causes of Concrete Disintegration," *Engineering News-Record*, Vol. 104, p. 735 (1930).

Reply to Merriman's paper on "Durability of Portland Cement," in Vol. 104, p. 62 (1930). Sugar solubility test proposed seems to be quite useful in determining resistance of different cements to sodium sulfate. No proof that behavior in sodium sulfate solution is any indication of performance which may be expected of cement under normal use.

(120) H. F. Gonnerman and G. W. Ward, "Determining the Suitability of Aggregates for Concrete Pavements," *Proceedings*, Assn. Highway Officials North Atlantic States, Sixth Annual Convention, p. 136 (1930).

(121) Stanton Walker, "Effect of Characteristics of Coarse Aggregates on Quality of Concrete," *Bulletin No. 5*, Nat. Sand and Gravel Assn. (1930). Abstracted in *Building Science Abstracts* (London), Vol. 3, No. 7, p. 243 (1930).

Test methods examined include variations of sodium sulfate tests, freezing and thawing test, etc.

(122) M. O. Withey and James Aston, "Johnson's Materials of Construction," John Wiley and Sons, New York City, Edition 7, pp. 246, 278, 498 (1930).

(123) M. O. Withey, "Some Long-Time Tests of Concrete," *Journal*, Am. Concrete Inst., Vol. 2, p. 547, February, 1931.

Absorption and alternate freezing-and-thawing tests.

(124) "Proposed Method of Test for Soundness of Fine and Coarse Aggregates by Use of Sodium Sulfate," Appendix 2 to Fifth Annual Report of Engineering and Research Division, *The National Sand and Gravel Bulletin*, Vol. 12, p. 41, February, 1931.

(125) G. W. Ward, "Letter to the Editor," *Civil Engineering*, Vol. 1, p. 553, March, 1931.

Discussion of article by H. F. Kriege in issue of November, 1930, p. 120, on testing aggregates for soundness.

(126) F. C. Lang and C. A. Hughes, "Discussion of Papers on Relation Between Durability of Concrete and Durability of Aggregates," *Proceedings*, Highway Research Board, p. 116 (1930).

(127) C. A. Hogentogler and E. A. Willis, "The Phenomena of Shrinkage and Its Effects upon the Integrity and Durability of Concrete Pavements," *Proceedings*, Highway Research Board, p. 164 (1930).

(128) C. H. Scholer, "The Durability of Concrete," *Proceedings*, Highway Research Board, Vol. 10, p. 132 (1930).

(129) H. F. Gonnerman and G. W. Ward, "Durability Studies of Aggregate and Concrete," *Proceedings*, Assn. State Highway Officials of North Atlantic States, p. 45 (1931).

## B. EFFECT OF MOISTURE IN HARDENED CONCRETE ON THE STRENGTH AND ELASTIC PROPERTIES

(130) J. W. Sussex, "Relative Strength of Wet and Dry Concrete," *Technograph* (1902).

(131) J. L. Van Ornum, "Effect on Tensile Strength of Cement of Removing Briquets from Water for Varying Lengths of Time Before Breaking," *Engineering News*, Vol. 51, p. 24 (1904).

(132) "Effect of Alternate Wetting and Drying on Strength of Concrete." Tests on 1:3 standard sand briquets from *Mitteilungen, Forschungsarbeiten* des Vereines Deutscher Ingenieure, Vols. 72 to 74 (1912). Reinforced Concrete Design, by Faber and Bowie, p. 19 (1912).

(133) S. Woodward and F. C. Young, "Effect of Moisture on Strength of Concrete," *Engineering News*, Vol. 69, p. 127, January 16, 1913.

Tests at Iowa State College show reduction of 40 to 50 per cent in strength of 1:3 mortar by wetting. Discussion of E. McCullough, p. 224.

(134) J. L. Van Ornum, "Effect of Saturation on Strength of Concrete," *Transactions*, Am. Soc. Civil Engrs., Vol. 77, p. 438 (1914).

(135) W. J. Schlick, "Effects on Concrete of Immersion in Boiling Water and Oven Drying," *Bulletin No. 59*, Iowa State College Engineering Experiment Station (1921).

(136) "Effects of Moisture on Concrete," *Génie Civil*, July 1, 1922.

(137) F. Anstett, "Influence of Drying on Resistance of Cement Briquets," Revue des Matériaux de Construction et Travaux Publics, No. 148, p. 1 (1922).

(138) W. K. Hatt, "Effect of Moisture on Concrete," *Public Roads*, Vol. 6, p. 14, March, 1925; also, *Proceedings*, Am. Soc. Civil Engrs., Vol. 51, p. 757, May, 1925; *The National Sand and Gravel Bulletin*, Vol. 6, p. 7, May 15, 1925; *Concrete*, Vol. 26, p. 192, June, 1925.

Cooperative tests by Purdue University and U. S. Bureau of Public Roads, for purpose of measuring maximum warping and surface deformation of concrete road slab resulting from non-uniform distribution of moisture as basis for estimating possible initial stresses; effect of moisture changes on modulus of rupture and compressive strength of concrete, and volume changes in concrete beams due to exposure and saturation.

(139) R. E. Stradling, "Effect of Moisture Changes on Building Materials," *Bulletin No. 3*, Department of Scientific and Industrial Research, London (1928).

Water can be present in building materials in 3 ways: (1) in chemical combination, where the water forms a definite chemical compound, for example, $Ca(OH)_2$, with other materials present, (2) as free water, where the association with other materials is such that the properties of the water remain unchanged, and (3) as "sorbed" water, where the association with a solid is in a condition intermediate between the two states just mentioned, the water being more firmly held to the solid than is free water, but not so held as to form a definite chemical compound.

## C. EFFECT OF SMOKE AND FLUE GASES ON CONCRETE

(140) F. T. Harvard, "Effect of Flue Gases and Moisture on Concrete," *Engineering News*, Vol. 53, pp. 11, 288, March 16, 1905.

(141) "Chemical Investigation of Concrete Bridges over Railroads," *Cement*, Vol. 9, p. 111, August, 1908.

(142) A. H. White, "Disintegration of Fresh Cement Floor Surfaces by Action of Smoke Gases at Low Temperature," *Engineering and Contracting*, Vol. 32, p. 27, July 14, 1909; also, *Cement Era*, August, 1909.

(143) "Effect of Locomotive Gases on Concrete," *Cement*, Vol. 11, p. 139, August, 1910.

(144) E. Willman, "Influence of Locomotive Gases on Fresh Mortar," *Chemical Abstracts*, Vol. 6, p. 1062, April 20, 1912.

(145) E. L. Conwell, W. M. Kinney, et al, "Effect of Fumes from Lignite Coal and from Wood on Cement Brick Chimneys," *Concrete-Cement Age*, December, 1914, p. 246.

(146) P. Rohland, "Action of Sulfur Compounds on Cement and Concrete," *Beton und Eisen*, Vol. 13, p. 241 (1914).

Pyrite, $H_2SO_4$ and $H_2SO_3$ have injurious effects on concrete. In railroad tunnels disintegration through gases containing sulfur results in formation of $CaH_2(SO_4)_2$, a colloidal plastic substance. This action proceeds quickly during early hardening period and when concrete is wet.

(147) A. T. Becker, "Effect of Boiler Stack Gases on Strength of Concrete," *Engineering and Contracting*, Vol. 45, p. 410, May 3, 1916.

(148) C. A. Newhall, "Disintegration of Concrete in Railway Tunnels," *Proceedings*, Pacific Northwest Soc. Engrs., October to November, 1916.

Tells of "sulfate disintegration" between sulfurous fumes from locomotive fuels and cement of the concrete; water necessary for the reaction.

(149) J. V. Schaefer, "Protecting Steel Bridges from Locomotive Blast," *Engineering News-Record*, Vol. 89, p. 968, December 7, 1922; *Canadian Engineer*, Vol. 43, p. 641, December 19, 1922.

Photographs show results of locomotive blast on gunite and other concrete. Amount of headroom and other factors considered.

(150) P. Levy, "Tests of Protecting Steel and Reinforced Concrete Structures from Locomotive Gases," *Ciment*, Vol. 30, p. 136, April, 1925.

(151) "Corrosion by Flue Gases," *Power*, Vol. 65, p. 87, January 18, 1927.

(152) E. C. Rack, "A Building Material to Resist Corrosive Atmospheres," *Chemical and Metallurgical Engineering*, Vol. 38, p. 157, March, 1930.

Use of transite, a material composed of portland cement and asbestos fiber in corrugated sheet form as building material to withstand action of unusually severe corrosive atmosphere. Use of material for $5\frac{1}{2}$ years proved that flue gases from incinerators in presence of moisture do not affect portland cement and asbestos fiber molded under tremendous pressure.

(153) K. T. Herrmann, "The Protection of Railway Tunnels from Destruction by Chemical Agents," *Revue des Materiaux de Construction*, No. 250, p. 274 (1930). Abstracted in *Building Science Abstracts*, Vol. 3, p. 390, November, 1930.

Disintegration of lime and mortar in railway tunnels from sulfur originating from smoke in contact with air and water.

## D. CONCRETE EXPOSED TO ALKALI SOILS AND WATERS

(154) M. Muth, "Decomposition of Cements by Water," *Tonindustrie Zeitung*, Vol. 33, p. 841 (1909); also, *Cement and Engineering News*, Vol. 22, p. 417 (1909).

(155) E. Burke and R. M. Pinckney, "Destruction of Hydraulic Cements by Action of Alkali Salts," *Bulletin No. 81*, Montana State Agricultural College (1910). Abstracted in *Industrial and Engineering Chemistry*, Vol. 3, p. 317 (1911).

Cause of frequent disruption of cement work by alkali water studied; samples of portland, natural and slag cement subjected to action of sodium sulfate, sodium carbonate and magnesium sulfate. Cements previously set with water and allowed to harden for several weeks were ground to pass a No. 100 sieve and leached with solutions for 5 to 11 weeks; no appreciable difference in behavior of three types of cements; sulfate solutions more active than sodium carbonate solution; confirm opinion generally held that magnesium sulfate in sea water is mainly responsible for disintegration.

(156) J. Y. Jewett, "Investigations of Modified Cements and of Alkali Action on Concrete of U. S. Reclamation Service," *Engineering and Contracting*, Vol. 38, p. 265, September 4, 1912; also, *Proceedings*, Internat. Assn. Testing Mats., Vol. 2, No. 13, Paper XX (1912); *Engineering Magazine*, Vol. 44, p. 267, November, 1912.

Outlines method of making investigations.

(157) P. H. Bates, A. J. Phillips, and R. J. Wig, "Action of Salts in Alkali Water and Sea Water on Cements," U. S. Bureau of Standards *Technologic Paper No. 12*, November 1, 1912.

(158) P. Rohland, "Action of Sulfur Compounds on Cement and Concrete," *Beton und Eisen*, Vol. 13, p. 241 (1914). Abstracted in *Chemical Abstracts*, Vol. 8, p. 3107 (1914).

Pyrite, $H_2SO_4$ and $H_2SO_3$ injure cement. In tunnels, etc., gases containing sulfur result in formation of $CaH_2(SO_4)$, a colloidal plastic substance.

(159) K. Steik, "Cause of Disintegration of Cements by Alkalies," Wyoming Station Report, p. 148 (1914); also, *Experiment Station Record*, Vol. 32, p. 787 (1914).

Extensive investigations with neat cement and mortar in alkali solutions. Effect on strength, setting, different brands of cement, waterproofing compounds, etc.

(160) W. Roeder, "Cements in Water Containing $H_2SO_4$," *Tonindustrie Zeitung*, Vol. 40, p. 664 (1916). Abstracted in *Chemical Abstracts*, Vol. 11, p. 2032 (1916).

Briquets of portland, iron and slag cements immersed in 1, 1.2, and 1.5 per cent $H_2SO_4$. Strongest portland cements showed formation of $CaSO_4 \cdot 2H_2O$ in 10 days. All were attacked; the slag cements slightly.

(161) A. G. Blackie, "Causes of Disintegration of Concrete," *Journal*, Engineering Inst., Canada, Vol. 1, September, 1918; also, *Engineering and Contracting*, November 27, 1918.

(162) M. Jacobson and J. G. De Keravenant, "Action of Chemicals on Concrete," *Industrie Chemique*, Vol. 7, p. 391 (1920). Abstracted in *Chemical Abstracts*, Vol. 15, p. 935 (1921).

Review of action of acids, acid salts, alkalies and alkali salts, coal-tar derivatives, fertilizers, oils and fats on concrete.

(163) W. Petry, "Action of Acids and Salts on Concrete," *Der Bauingenieur*, Vol. 15, p. 12 (1920); also, *Wasser und Abwasser*, Vol. 15, p. 273 (1921). Abstracted in *Chemical Abstracts*, Vol. 16, p. 3740 (1922).

Waters low in salt content dissolve $Ca(OH)_2$ from concrete. Alkali carbonates and nitrates and $CaCl_2$ harmless; $MgSO_4$ and $MgCl_2$ very destructive. Investigations include effect of NaCl, KCl, etc.

(164) A. Bruce, "Action of Sulfur on Cement in the Tropics," *Journal*, Soc. Chemical Industry, Vol. 40, p. 240 (1921). Abstracted in *Chemical Abstracts*, Vol. 16, p. 325 (1922).

Disintegration of cement mortar joining earthenware sewer pipes and analytical data. Disintegration caused by sulfates. One analysis shows CaO, 27.8 per cent; $SO_3$, 39.2 per cent; $\frac{CaO}{SO_3}$, 0.71.

(165) W. G. Atwood and A. A. Johnson, "The Disintegration of Cement in Sea Water," *Transactions*, Am. Soc. Civil Engrs., Vol. 87, p. 204 (1924).

A résumé of investigations of the effect of sea water on concrete and an exhaustive bibliography of the literature on the subject.

(166) C. J. Mackenzie and T. Thorvaldson, "Differentiation of Action of Acids, Alkali Waters, and Frost on Normal Portland Cement Concrete," *The Engineering Journal*, Vol. 9, p. 79, February, 1926.

(167) "Specifications for Construction of Concrete Buildings in Swamps, Swamp Waters and Waters of Similar Composition," Ernst and Son, Berlin (1927).

Concrete construction in water containing acids and other substances injurious to concrete.

(168) R. B. Young, "Requirements for Durable Concrete as Observed from Structures in Service," *The Engineering Journal*, Vol. 11, p. 161, March, 1928.

(169) R. Wilson and A. Cleve, "Brief Summary of the Effect of Sulfate Soils and Waters on Concrete," *Report*, Director of Research, Portland Cement Assn., p. 70, (1928). Abstracted in ***Building Science Abstracts***, Vol. 2, p. 260, July, 1929.

About 2000 concrete cylinders 10 by 24 in. covering wide range of concrete quality (lean to rich mixes, wet to dry consistencies, poorly to well-graded aggregates, etc.) placed in soils and waters in different localities known to offer abnormally severe conditions, suitable for accelerated tests. Eight inspections made covering 7 years. Disintegration in part due to frost action. Close relation found to exist between resistance to sulfate attack and 28-day compressive strength of workable, plastic concrete. Use of surface coatings of which boiled linseed oil proved best.

(170) T. Thorvaldson, D. Wolochow and V. A. Vigfusson, "Study on Action of Sulfates on Portland Cement," ***Canadian Journal of Research***, Vol. 1, p. 273, September, 1929; p. 359, October, 1929; p. 385, November, 1929.

Part I. The Use of the Expansion Method in the Study of the Action of Sulfates in Portland Cement Mortar and Concrete.

Part II. Steam-Curing of Portland Cement Mortar and Concrete as a Remedy for Sulfate (Alkali) Action.

Part III. Effect of the Addition of Silica Gel to Portland Cement Mortars on Their Resistance to Sulfate Action.

(171) D. G. Miller, "Strength and Resistance to Sulfate Waters of Concrete Cured in Water Vapor at Temperatures Between 100 and 350° F.," ***Proceedings***, Am. Soc. Testing Mats., Vol. 30, Part II, p. 636 (1930); also, ***Rock Products***, Vol. 33, p. 104, July 5, 1930. Abstracted in ***Building Science Abstracts***, Vol. 3, p. 389, November, 1930.

Variations of temperature of vapor had little effect on strength, but marked effect on resistance to sulfate waters. Not much difference noted up to 190° F., but from 212° F. up, definitely increased resistance to action of sulfate waters, more especially in range from 212 to 260° F.

(172) H. A. Holt, "Cement Complaints," ***Cement and Cement Manufacture*** Vol. 3, p. 567, April, 1930.

Part 2. Causes of disintegration; effect of coal, sulfides in aggregate, sulfuric anhydride in aggregate, sugar, aquatic vegetable growths.

(173) A. Kleinlogel, "Concrete Deterioration," ***Beton und Eisen***, Vol. 29, p. 144 (1930).

Chemical, mechanical, and other action of air, water, acid, alkalies, oils, vapors, soils, etc., on cement, mortar and concrete.

# BIBLIOGRAPHY ON WEATHERING OF CONCRETE MASONRY UNITS

### BY F. O. ANDEREGG[1]

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(175) Richard Grün, "Der Beton," Julius Springer, Berlin (1926).

(176) H. L. Childe, "Manufacture and Uses of Concrete Products and Cast Stone," Concrete Publications, London (1926).

(177) A. B. Searle, "Cement, Concrete and Bricks," Second Edition, Constable and Co., Ltd., London (1926).

(178) W. K. Harris, "Concrete Products, Their Manufacture and Uses," International Trade Press, Chicago (1925).

(179) Report of Committee P-1 on Tests on Concrete Building Tile, *Proceedings*, Am. Concrete Inst., Vol. 23, p. 691 (1927).

Strengths were constant for a given fineness modulus within 15 per cent. Generally compressive strength is to be correlated with high unit weight and low absorption and the importance is emphasized of controlling the water to secure the denseness desired.

(180) R. E. Davis, "Volumetric Changes in Cements, Mortars and Concretes Due to Causes Other than Stress," *Journal*, Am. Concrete Inst., Vol. 1, p. 407 (1930).

A review of the literature with bibliography.

(181) L. A. Falco, "Recommended Practices in the Use of Cast Stone," *Journal*, Am. Concrete Inst., Vol. 1, p. 760 (1930).

(182) P. M. Woodworth, "Recommended Practice for the Manufacture of Building Block and Concrete Tile," *Journal*, Am. Concrete Inst., Vol. 1, p. 49 (1930).

Recommendations include: addition of sufficient water to produce maximum strength; mixing at least five minutes; use of vibration; and curing with initial moist curing followed by sufficient drying to remove shrinkage.

(183) G. M. Williams, "Disintegration of Concrete," *Journal*, Am. Concrete Inst., Vol. 1, p. 41 (1929).

Emphasizing alkali salt disintegration.

(184) Report of Special Committee of the Cast Products Assn. on Crazing in Cast Stone, *Cement Lime Gravel*, Vol. 4, p. 189 (1930).

Steam curing for two days helps to cure crazing, while tooled or scrubbed surfaces are practically free from crazing.

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[1] Consulting Specialist on Building Materials, Pittsburgh, Pa.

**(185) F. O. Anderegg, "The Construction of Water-Tight Brick Masonry," *Journal*, Am. Ceramic Soc., Vol. 13, p. 315 (1930).**

An Experimental Study of Brick Masonry with Special Reference to Watertight Construction. (*Bulletin*, Am. Face Brick Assn., Chicago (1931).) (*a*) Workable mortar is needed. (*b*) The mortar must have a certain amount of strength, but the strength of the mortar as such is usually less important than its adhesion to the units. The strength should not be so great as to prevent the mortar from having sufficient flexibility to allow it to adjust itself to the wall movements. (*c*) The mortar had best be waterproofed with a properly distributed stearate waterproofer. (*d*) The mortar should be thoroughly mixed, best from lime putty and cement in nearly equal parts by volume with six of sand, and by machine in a batch mixer. The mortar should stand at least half an hour before using but should not be used after three hours. (*e*) The joints of the facing should be full and the mortar, especially in the head joints, should be well compacted to get good adhesion. (*f*) The surface of the joints should be finished with a special tool so as to bring the mortar in close contact with the units on both sides of the joint and after the mortar has reached the initial set, about two hours. (*g*) Hollow block are especially suitable for parapet walls and chimneys because of the internal ventilation. (*h*) Masonry should be protected from ground and other sources of moisture by waterproof membranes which may be of metal, impregnated fabrics or of trowelled-on mortar containing properly dispersed stearate. (*i*) Water entering a masonry wall from leaking roofs, gutters or downspouts often contains sulfuric acid which attacks the lime to form calcium sulfate which disintegrates the masonry by crystal pressure.

**(186) F. O. Anderegg, "The Mechanism of the Corrosion of Portland Cement Concrete with Special Reference to Crystal Pressure," *Proceedings*, Am. Concrete Inst., Vol. 25, p. 332 (1929).**

A review of the methods by which concrete can be corroded including: the solvent action of nearly pure water, of highly carbonated, and of winter rain water for lime; the effect of presence of salts, especially under the influence of temperature changes in the presence of moisture.

**(187) P. H. Bates, "Crazing on Concrete Products," *Proceedings*, Am. Concrete Inst., Vol. 21, p. 126 (1925).**

Crazing is connected with formation of a surface layer different from the body.

**(188) P. H. Bates and C. H. Jumper, *Proceedings*, Am. Concrete Inst., Vol. 24, p. 179 (1928).**

The crazing of stucco panels depends chiefly upon curing and exposure.

**(189) Minoru Hamada, "Thermal Constants and Strengths with Light Weight Aggregate," *Journal*, Engineering Faculty, Tokyo Imperial University, Vol. 17, p. 201 (1928).**

Relationship between heat conductivity, $k$, and voids, $v$, was found to be $k = \frac{0.005}{0.5^v}$ approximately. It is readily possible to reduce the conduction to half or a third that of common concrete.

**(190) R. F. Havlik, "The Wet Cast Process of Manufacturing Concrete Stone," *Proceedings*, Am. Concrete Inst., Vol. 23, p. 213 (1927).**

Different types of molds are described, as well as methods of curing and finishing. The product should resist staining and should weather well.

(191) H. Kreuger, "Investigation of Climatic Conditions on the Exterior of Buildings," *Ingeniörsvetenskapsakademien*, No. 24, pp. 68, 92 (1923).

Sufficient pore space unfilled by water should be present to take up the expansion of the water on freezing. Walls built of the hardest burned brick and mortar high in portland cement do not seem to withstand weathering as well as materials of greater absorption.

(192) F. M. Lea, "Investigation of Breeze and Clinker Aggregate," *Building Research Bulletin, No. 7.* H. M. Stationery Office (1929).

Certain coals high in oxygen content and with high absorptive power for methylene blue, also absorb more oxygen readily and tend to expand early in the life of the concrete to cause damage. The aggregate should be fairly well sintered and should not contain more than 0.45 per cent sulfide sulfur and not more than 0.4 per cent sulfate sulfur.

(193) G. F. Loughlin, "Qualifications of Different Kinds of Natural Stone for Concrete Aggregate," *Proceedings*, Am. Concrete Inst., Vol. 23, p. 319 (1927).

Sound and unsound aggregates described.

(194) A. W. Scheer, "Light Weight Aggregate in the Manufacture of Concrete Masonry Units," *Proceedings*, Am. Concrete Inst., Vol. 24, p. 436 (1928).

Experience with haydite is reported to be satisfactory as to strength, lightness and durability.

(195) E. O. Sweetser, "Durability of Concrete Exposed to the Weather," Washington University, St. Louis, Mo.

The effect of waterproofing, mixing and methods of curing, employed to reduce absorption, on the strength of concrete.

(196) John Tucker, Jr. and G. W. Walker, "The Physical Properties of Cast Stone," *Proceedings*, Am. Concrete Inst., Vol. 25, p. 501 (1929).

Results given from tests on commercial cast stone for compressive and flexural strengths, for absorption, and for weathering as determined by freezing and thawing.

(197) A. H. White, A. A. Vilhelm and A. O. L. Christensen, "Crazing in Concrete and the Growth of Hair Cracks into Structural Cracks," *Proceedings*, Am. Concrete Inst., Vol. 24, p. 190 (1928).

(198) Raymond Wilson, "Limitations of the Absorption Test for Concrete Products," *Proceedings*, Am. Concrete Inst., Vol. 25, p. 522 (1929).

The quality of concrete products does not seem to correlate well with the absorption as usually determined; the absorption rate is probably more important than the total absorption.

(199) P. M. Woodworth, "Concrete Masonry Units Cured with High Pressure Steam," *Journal*, Am. Concrete Inst., Vol. 1, p. 504 (1930).

Units cured in high pressure steam were stronger in two days than when cured moist or in air or with low pressure steam, even at 28 days. High pressure steam curing greatly reduced the shrinkage of concrete masonry units when laid up wet and if air dried the shrinkage was very small.

# BIBLIOGRAPHY ON THE WEATHERING OF STRUCTURAL CLAY PRODUCTS[1]

By Douglas E. Parsons[2]

(200) W. Olschewsky, "Die Ursachen der verwitterung bei Verblindsteinen und Terrakotten," Halle (1885).

States that weathering is due to impurities in the raw material and unsuitable methods of manufacture. Prolonged firing at high temperatures was found necessary for the production of durable ware. Some of the soluble salts in bricks and terra cotta tend to cause disintegration. Chemical analyses of salt deposits are given.

(201) J. C. Jones, "The Relation of Hardness of Brick to Their Resistance to Frost," *Transactions*, Am. Ceramic Soc., Vol. 9, p. 528 (1907).

Discusses the pore structure of brick and the effect of firing on structure, strength and rigidity. Underburned bricks sometimes contain large amounts of salts which may cause disintegration and slaking when the bricks are placed in water, even after firing at temperatures as high as 1100° F. Hard-burned brick are not subject to this type of disintegration because the salts are combined or changed to a harmless form at high temperatures. Thermal stresses usually are too small to cause failures. Crystallization of salts may cause some decay of sound brick, but the most destructive agency of weathering probably is frost. Since the freezing point of water is only lowered about 0.0009° F. for each increase of pressure of 1 lb. per sq. in., large pressure may be developed by frost action. Pores are only partially filled, however, and a portion of the water is forced out of the body as soon as pressures are formed. Also no large pressures can form in a pore unless it is more than 0.9 full. The most damage is done by the freezing of water in the smallest pores, because the water in these freezes last. Since the coefficient of expansion of ice is several times that of brick, ice can damage brick only at the time of its formation. A rate of absorption test is suggested as a measure of size of pores. No relation was found between loss on freezing and thawing and either the porosity or the hardness. On the average the bricks with coarse pores showed the greatest resistance.

(202) Heinrich Ries, "Absorption Tests on Wisconsin Brick," *Transactions*, Am. Ceramic Soc., Vol. 9, p. 693 (1907).

The paper gives data on the absorption and strength of several makes of brick. In a discussion, Professor Wheeler calls attention to factors other than porosity that affect the frost resistance of brick.

(203) J. W. Cobb, "Note on Disintegration by Crystallization and Freezing," *Transactions*, Am. Ceramic Soc., Vol. 11, p. 65 (1909).

The author calls attention to the destructive effect of the crystallization of sodium sulfate and thiosulfate. The forces in this case are different from those caused by the expansion of water to ice.

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[1] Publication approved by the Director of the Bureau of Standards of the U. S. Department of Commerce.

[2] Chief, Masonry Construction Section, U. S. Bureau of Standards, Washington, D. C.

(204) J. A. Van der Kloes, "Manual for Masons, Bricklayers, Concrete Workers and Plasterers," London (1914).

(205) K. Dümmler, "Handbuch der Ziegel Fabrication," p. 516 (1914).

Dümmler describes the following methods for testing the resistance of clay products to weathering: (1) frost action, (2) crystallization of sodium sulfate, (3) crazing and (4) chemical action.

(206) W. L. Howat, "The Effect of Saturated Sodium Sulfate Solution upon the Structure of Clay Burned to Different Temperatures," *Transactions*, Am. Ceramic Soc., Vol. 17, p. 249 (1915).

Howat found that the resistance of brick to the crystallization test increased as the ware was fired to higher temperatures.

(207) H. F. Staley, "A Comparison Between Sodium Sulfate and Freezing Tests for Drain Tile," *Transactions*, Am. Ceramic Soc., Vol. 18, p. 642 (1916).

Principal data and conclusions are given in abstract reference No. 208.

(208) Edward Orton, Jr., "A Comparison Between the Absorption, Crushing Strength and Resistance to Artificial Freezing of Some Ohio Building Bricks," *Transactions*, Am. Ceramic Soc., Vol. 18, p. 686 (1916).

Principal data and conclusions are given in abstract reference No. 209.

(209) W. J. Schlick and H. F. Staley, "An Investigation of Tests of Iowa Shale Drain Tile," *Bulletin No. 49*, Iowa State College Engineering Experiment Station (1918).

The paper gives the results of strength, absorption, freezing and sodium sulfate tests on drain tile. It was found that the absorption was more closely related than the strength to frost resistance. The theory of the sodium sulfate test is discussed and a procedure is recommended. It is pointed out that saturated solutions are not well adapted to such tests because of variations in solubility with temperature. A 15-per-cent solution was used in the tests. Results of sodium sulfate tests were not concordant with those of freezing tests over a wide range in absorption. Between the limits of 7 to 11 per cent absorption, one sulfate treatment approximately equaled three freezings.

(210) Edward Orton, Jr., "A Study of the Proposed A.S.T.M. Tentative Specifications for Building Brick and a Correlation of Their Requirements with Sodium-Sulfate Treatment and Actual Freezing," *Proceedings*, Am. Soc. Testing Mats., Vol. XIX, Part I, p. 268 (1919).

Using a 14 per cent solution (at 21° C.) the author compared the results of the sodium sulfate test with freezing and thawing. A fair correlation was found between the results of the two tests, 4 crystallization tests being equivalent to something more than 25 freezings and thawings. The details of the tests were well controlled and the reasons for control are discussed.

(211) W. J. Scott, "Tile Fungus," *British Clayworker*, Vol. 29, p. 138 (1920).

States that a fungus growth may take place in the pores of tile.

(212) H. A. Plusch, "Some Causes of the So-called Disintegration of Terra Cotta, Brick and Allied Ceramic Materials," *The Ceramist*, Vol. 1, p. 65 (1921).

The paper contains a discussion of the effects of temperature variations and of settlement and vibrations of buildings. Disintegration is attributed largely to laminations, poor mortar joints, soluble salts in ware and impervious surfaces on porous bodies.

(213) E. C. Hill, "Fire Cracking of Terra Cotta," *Journal*, Am. Ceramic Soc., Vol. 5, p. 299 (1922).

(214) E. C. Hill, "The Effect of Some Fluxes on the Absorption and Transverse Strength of Terra Cotta," *Journal*, Am. Ceramic Soc., Vol. 5, p. 832 (1922).

The effect of body composition on physical properties is discussed and data are given which indicate that cracks developing during weathering may be due to too rapid cooling after firing and the presence of sand in the clay.

(215) I. Furlong, "Alkali Attack on Concrete Roads and Building Brick. A Study of Disintegration in Glenn County, California," *Engineering News-Record*, Vol. 89, p. 64, July 13, 1922.

Alkali consisting chiefly of sodium sulfate, caused the disintegration of concrete roads and of the mortar and underburned bricks in the lower portions of building walls.

(216) H. Kreuger, "Investigations of Climatic Action on the Exteriors of Buildings," *Transactions*, Royal Swedish Inst. for Scientific Industrial Research, No. 24 (1923).

Reports on an exhaustive study of the weathering of building materials, including such subjects as the nature of weathering, essential physical properties of materials, structural features affecting weathering and laboratory weathering tests. Many buildings examined were defective as to flashing and other protective features. Materials which are not greatly weakened by becoming saturated and which ordinarily do not absorb a volume of water greater than 85 per cent of the total pore space usually have a high resistance to frost action.

(217) H. D. Foster, "Strength, Absorption and Freezing Resistance of Hollow Building Tile," *Journal*, Am. Ceramic Soc., Vol. 7, p. 189 (1924).

Tiles from 25 different sources were tested for strength, absorption, density, modulus of elasticity and frost resistance. Color was not indicative of other qualities. Absorption was closely related to weight per unit volume of material and roughly related to frost resistance.

(218) Report of Committee C-10 on Hollow Building Tile, *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part I, p. 411 (1924).

Data are given on physical properties of hollow tiles from 25 sources. These included results of tests for compressive strength, modulus of elasticity, absorption, porosity, density and frost resistance. Tentative specifications, based largely on the test data are proposed.

(219) Hewitt Wilson, "Monograph and Bibliography on Terra Cotta," *Bulletin*, Am. Ceramic Soc., Vol. 9, p. 94 (1926).

An exhaustive review of the literature pertaining to terra cotta. Discusses manufacturing problems, properties of terra cotta affecting durability and structural features of terra cotta wall construction.

(220) C. O. Christenson, "Discussion of Specification Requirements for Common Brick," *American Architect*, July 5, 1926, p. 23.

Data are given on the strength and absorption properties of brick and the relations between these properties are shown. The results of 300 cycles of freezing and thawing of three types of brick are reported.

(221) R. L. Jack, "Pitting of Tiles and Bricks at the Seaside," *Bulletin No. 12*, Geological Survey of South Australia, p. 16 (1926).

(222) A. P. Laurie and J. Milne, "The Evaporation of Water and Salt Solutions from Surfaces of Stone, Brick and Mortar," *Proceedings*, Royal Soc. Edinburgh, Vol. 47, Part I, No. 4, p. 52 (1926).

Experiments on wall surfaces show that portion of wall where evaporation is most rapid attracts water from the slower drying portions. Hence if brick dry more rapidly than mortar, moisture and soluble salts will be drawn from the mortar into the brick; if mortar dries most rapidly the salts will concentrate in the mortar. Hence too dense a mortar accelerates decay of stone and brick.

(223) O. Kallauner, "Injurious Effects of Nodules in Brick Products," *Transactions*, Ceramic Soc. (British), Vol. 26, No. 2, p. 90 (1926).

Harmful effects of nodules may be determined by steaming for 4 to 5 hours.

(224) A. P. Laurie, "The Decay of Modern Brickwork," *British Clayworker*, Vol. 35, p. 280 (1927).

Attributes disintegration to effect of $CaSO_4$ formed by the $SO_2$ and $H_2O$ from the air acting on the cement in the mortar.

(225) J. W. McBain and John Ferguson, "On the Nature of the Influence of Humidity Changes upon the Composition of Building Materials," *Journal of Physical Chemistry*, Vol. 31, No. 14, p. 562 (1927).

A systematic study was made of the effects on building materials of changes in humidity and an explanation is given for the changes in moisture content and volume.

(226) O. Kallauner, "Tentative Methods and Specifications for Ceramic Products," *Zement*, Vol. 16, p. 928 (1927).

Discusses specifications and tests for (1) deleterious lime nodules, (2) permeability, (3) absorption, (4) soluble content and (5) chemical resistance.

(227) J. Obata, "The Discrimination of the Quality of Brick by Means of Sound," *Journal*, Franklin Inst., Vol. 203, p. 647 (1927).

The author used a condenser microphone to investigate the relationship between the pitch of the sound produced by striking the brick with a light hammer and the quality of the brick. A linear relation was found between the pitch and the square root of the ratio of Young's modulus to density. Relations were found between pitch and compressive strength and the effect of absorption were also investigated.

(228) W. Kraemer, "The Setting of Terra Cotta and Its Inspection by Architects," *American Architect*, Vol. 131, p. 551 (1927).

Discussed the features of construction affecting the durability of terra cotta walls.

(229) Report of the Building Research Board for the Year 1926, Department of Scientific and Industrial Research, London (1927).

The report reviews the work of the Board during 1926. Weathering effects are considered under the headings (1) chemical attack, (2) biological attack, (3) movements due to temperature and moisture changes and (4) frost. Data are given on the moisture expansion of various masonry materials; that of clay products was less than most other materials. Found that Kreuger's "coefficient of saturation" was a fair but not entirely reliable measure of frost resistance. It is suggested that the strength, modulus of elasticity and expansion during freezing are factors affecting the liability to damage. Observations indicate that the type of mortar used has a marked influence on the durability of masonry units.

(230) Harry Spurrier, "Some Fundamentals of Terra Cotta," *Journal*, Am. Ceramic Soc., Vol. 9, p. 773 (1926).

(231) Harry Spurrier, "Some Observations on Terra Cotta Physics," *Journal*, Am. Ceramic Soc., Vol. 10, p. 686 (1927).

The author describes experiments indicating that freezing and thawing of terra cotta specimens immersed in water increased both the porosity and permeability and that no osmotic pressure developed with a terra cotta membrane. The second paper deals with volume changes due to changes in temperature and moisture content. Specimens contracted when first immersed in water. After heating to 207° F. and then allowed to cool a large expansion took place near the end of the cooling period.

(232) W. A. McIntyre, "Factors Governing the Durability of Clay Building Materials," *British Clayworker*, Vol. 37, p. 195 (1928).

Gives brief review of the literature on the weathering of clayware in relation to moisture expansion, differential expansion, thermal expansion, abrasion, rupture by freezing, rupture by crystallization of salts, osmotic pressure, porosity, absorption, saturation coefficient and chemical action. He finds that efflorescence consisted largely of sulfates of sodium, calcium, magnesium and potassium.

(233) Report of the Building Research Board for the Year 1927, Department of Scientific and Industrial Research, London (1928).

In the review of investigative work of the Board on the weathering of masonry, the mechanism of biological and chemical attack of stone is discussed. Data are given on porosity, pore structure and capillary properties of stones. An investigation of efflorescence on brickwork is described and chemical analyses of salt deposits taken from masonry are given. The transfer of soluble salts between two different materials in contact is discussed. Calculations indicate that the temperature gradients within a wall are not apt to cause stresses of high intensities. Experiments on volume changes during freezing and thawing are described, the results indicating that the expansions during freezing may be related to frost resistance.

(234) J. A. Van der Kloes, "Mortar, Its Importance in the Life of Brickwork," *Rock Products*, Vol. 31, p. 60, September 29, 1928.

The author states that osmotic pressure may cause the disintegration of clay products. He suggests the addition of pozzuolanic materials to mortars as a preventive.

(235) W. J. Schlick, "Tests of Clay, and Concrete Load-Bearing Pipe," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 635 (1928).

Discusses strength and absorption tests of pipe and states that the freezing-and-thawing test is the only direct criterion of durability.

(236) W. A. McIntyre, "Factors Governing the Durability of Clay Building Materials," *Transactions*, Ceramic Soc., Vol. 28, p. 101, March, 1929.

The paper describes a comprehensive study of the weathering of clay products including a review of the literature, examinations of numerous structures and laboratory tests. Evidence shows that biological action is of minor importance in the weathering of clay wares. The chief cause of disintegration was the crystallization of soluble salts in the body of the material.

(237) R. E. Stradling, "Effects of Moisture Changes on Building Materials," *Building Research Bulletin No. 3*, Department of Scientific and Industrial Research, London (1928).

Points out the ways that water may be present in a building material, namely, (1) in chemical combination, (2) as free water and (3) as "sorbed" water. The effects produced upon different materials by moisture in each of these three conditions are considered with special reference to the disintegrating action.

(238) H. G. Schurecht, "Methods for Testing Crazing of Glazes Caused by Increases in Size of Ceramic Bodies," *Journal*, Am. Ceramic Soc., Vol. 11, p. 271 (1928).

Describes an investigation of the cause of crazing that sometimes takes place after exposure to weather. Data from repeated thermal expansion measurements showed that the weathered bodies had expanded. Heating to 720° C. during the tests drove off combined water. Similar expansions of certain fired ceramic bodies were produced by steaming in an autoclave with 150 lb. per sq. in. pressure. A correlation was found between expansion due to weathering and that produced by autoclave treatment. Bodies showing large expansions usually gave high values also in tests for ignition loss and solubility in sulfuric acid.

(239) G. A. Bole, "The Drain Tile Industry of Ohio," *The Clayworker*, Vol. 90, p. 286 (1928).

From the results of a survey of materials and products it is concluded that pipe made from surface clays are more easily affected by freezing and thawing than those made from shale or shale-fireclay mixtures. Pipes containing lime are relatively less resistant to weathering than those free from such impurities. Laminations may reduce weather resistance greatly. Pipes of average quality are not affected by 30 or more freezings.

(240) "Tiles in Cold Weather," *Revue des Matériaux de Construction* (1929).

States that the primary cause of vulnerability of roofing tile to frost action is under-firing. Severe frosts in 1929 caused damage to individual tiles that were underfired but did not affect others from the same kiln. Porous tiles if properly fired and free from severe laminations appear to resist frost action.

(241) H. G. Schurecht and G. R. Pole, "Effect of Water in Expanding Ceramic Bodies of Different Compositions," *Journal*, Am. Ceramic Soc., Vol. 12, No. 9, p. 596 (1929).

Specimens of fired clay were soaked in water at 20° C. for periods up to two weeks, then boiled for 4 hours. Others were subjected to steam at 75, 100, 125 or 150 lb. per sq. in. pressure for one hour. The changes in length produced by each of these treatments was observed. A single autoclave treatment caused more expansion that a soaking. The expansion after steaming for one hour with 150 lb. per sq. in. pressure ranged from 0 to 0.16 per cent. Data showing effect of body composition on moisture expansion are given.

(242) E. Blanck, "Soil Science—Part II. Weathering Science and Its Climatic Basis," (Handbuch der Bodenlehre. Band II. Verwitterungslehre und ihre klimatalogischen Grundlagen), Julius Springer, Berlin (1929).

(243) W. A. McIntyre, "Investigations into the Durability of Architectural Terra Cotta and Faïence," *Special Report No. 12*, Department of Scientific and Industrial Research, Building Research Board of Great Britain (1929).

The causes of disintegration are enumerated and examples of certain types are illustrated. Views of old structures are given and the behavior of the terra cotta facing is discussed. The literature relating to the weathering of terra cotta is reviewed and discussed. The experimental work included detailed studies of pore structure, measures of strength and elastic properties and of moisture and thermal expansions, abrasives, freezing and sodium sulfate tests, porosity, absorption, etc., and chemical analyses. The results indicated that the chief agent of decay was the crystallization of salts within the pores, the salts being formed by the action of the atmosphere on certain constituents of the terra cotta. This type of decay occurred only with underburned ware. Laminations parallel to the surface tended to accelerate decay.

(244) J. W. McBurney, "The Water Absorption and Penetrability of Brick," *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part II, p. 711 (1929).

Gives data on the water absorption, rates of absorption and penetrability of brick and discusses the effects of absorptive properties on the resistance of brick to frost action and water transmission through brick masonry. Absorptive properties were not closely related to durability.

(245) H. Gilardoni, "Action of Freezing on Fire-Clay Products," Revue des Matériaux de Construction, Part II, p. 145-B (1929).

(246) H. Gilardoni, "Frost Rending of Terra Cotta, etc.," *La Ceramique*, Vol. 32, p. 113 (1929).

(247) Report of the Building Research Board for the Year 1928, Department of Scientific and Industrial Research, London (1929).

The report gives an account of progress made on several investigations relating to building materials and building construction. Under the general topic of weathering the following subjects are treated: Efflorescences, their causes, effects and prevention; factors influencing the distribution of moisture in porous materials; selective decay; bacteria, lichens and mosses as agencies in stone decay; the action of calcium sulfate and atmospheric pollution; tests for estimating resistance to weathering.

(248) W. Miehr, H. Immke and J. Kratzert, "Open and Closed Pores in Ceramic Ware," *The Ceramic Age*, Vol. 14, p. 4 (1929).

Only a small percentage of pores are closed, the maximum found in tests being 7 per cent.

(249) H. Hirsch, "Recent German Scientific Investigations in the Brick Industry," *Tonindustrie Zeitung*, Vol. 54, No. 82, p. 1303; No. 84, p. 1330; and No. 86, p. 1356 (1930).

The author describes investigations of the combination of lime with silica, the formation of sulfates and methods of fixing sulfur compounds. Water-tightness and permeability to air of roofing tiles were found to be related. The compressive strengths of cubes and cylinders cut from brick were found to be greater than values obtained on whole brick. Rapid alternations of freezing and thawing were more severe than slower ones. For testing the weather resistance of brickwork, a method for simulating conditions of driving wind and rain is illustrated. Water penetration usually occurred through mortar joints. Values are given of results of tests, made over a period of years, of water absorption, compressive strength, frost resistance and permeability of brick and roofing tiles. Heat conductivities of bricks and highly porous blocks are compared.

(250) L. F. Cooling, "Contributions to the Study of Efflorescence. The Evaporation of Water from Brick," *British Clayworker*, Vol. 38, No. 453, p. 384 (1930).

The author discusses decay caused by the crystallization of soluble salts in brickwork. Descriptions are given of experiments on the drying of brick. The mechanism of drying is discussed with special reference to the factors affecting the movements of water soluble salts and the position at which crystallization occurs. The drying rates were found to be greatly influenced by physical structure. The transfer of salts between different materials in contact is discussed and it is suggested that the direction of transfer and the position of crystallization depend on the capillarity of the materials, which depends upon pore sizes.

(251) O. J. Whittemore, "Absorption: Its Relation to Durability," *Journal*, Am. Ceramic Soc., Vol. 13, p. 80 (1930).

Discusses results of freezing-and-thawing tests of clay tile. It was found that absorption requirements intended as measures of durability should not be the same for all products but should be determined for each type of clay.

(252) O. Graf, "Permanency of Building Materials and of the Elements of Construction" (1930).

(253) O. Graf and H. Goebel, "Protection of Structures Against Chemical and Physical Attack," Wilhelm Ernst und Sohn, Berlin (1930).

(254) A. Simon and W. Vetter, "Systematic Investigations of the Behavior of Soluble Salts When Burned in Clays, a Contribution to the Study of Florescenses," Part I, *Berichte*, Deutsche Keramische Gesellschaft, Vol. 11, p. 12 (1930). Part II. *Zeitschrift für anorganishe Chemie*, Vol. 186, p. 225 (1930).

The authors describe a comprehensive investigation of the effects of body composition and firing treatment on the tendency of claywares to effloresce.

(255) J. W. Whittemore and P. S. Dear, "Physical Properties of Common and Face Brick Manufactured in Virginia," *Bulletin No. 6*, Virginia Polytechnic Inst. (1930).

Bricks from 29 plants were tested for strength, absorption and resistance to 10 freezings and thawings. The laminar structure of the brick appeared to have a marked influence on the results of the freezing tests.

(256) H. E. Weightman, "The Weather-Ometer," *The Ceramic Age*, Vol. 16, p. 267 (1930).

An apparatus is described which, in a short period, may be used to determine whether a material has combined resistance to freezing, thawing, temperature changes, moisture and light.

(257) W. Steger, "The Measurement of Strains in Fired Ceramic Materials," *Berichte*, Deutsche Keramische Gesellschaft, Vol. 11, p. 124 (1930).

Describes a method of measuring strains in clayware caused by differences in thermal expansion between the glaze and the body.

(258) "Change in Volume of Burned-Clay Bodies During Use," U. S. Bureau of Standards *Technical News Bulletin No. 153*, p. 1, *No. 156*, p. 34, and *No. 160*, p. 78 (1930).

The article gives results of moisture expansion during exposure to weather and expansion produced by autoclave treatment at 150 lb. per sq. in. steam pressure.

(259) L. A. Palmer and J. V. Hall, "Some Results of Freezing-and-Thawing Tests Made With Clay Face Brick," *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part II, p. 767 (1930).

The authors describe freezing-and-thawing tests of 100 cycles on samples of bricks from four sources. It was found that freezing and thawing reduced compressive strength more than bending strength. Absorption was increased in most cases. Losses were greatest for bricks from each source having highest absorption, but absorption was not a reliable criterion of resistance to frost action when comparing bricks from different sources.

(260) J. O. Everhart, "Some Simulative Service Tests for Glazed Building Materials," *Journal*, Am. Ceramic Soc., Vol. 13, No. 6, p. 404 (1930).

Glazed bricks obtained from one source were subjected to one of the following tests: (1) Absorption after drying at 110° C.; (2) Absorption after subjecting to steam pressure of 150 lb. per sq. in. for one hour; (3) absorption after freezing and thawing 14 times while immersed; (4) absorption after freezing in air and thawing in water; (5) immersion in saturated solution of sodium sulfate for 24 hours and drying for 72 hours. States that autoclave treatment may cause differential expansion between body and glaze; freezing and thawing tends to cause failures of some bodies.

(261) H. G. Schurecht and G. R. Pole, "Method of Measuring Strains Between Glazes and Ceramic Bodies," *Journal*, Am. Ceramic Soc., Vol. 13, p. 369 (1930).

A method is described for measuring strains in clayware produced by differential thermal expansion or moisture expansion of body and glaze.

(262) R. G. Mills, "The Use of the Autoclave in Testing Ceramic Products for Resistance to Crazing," *Journal*, Am. Ceramic Soc., Vol. 13, No. 12, p. 903 (1930).

The paper describes the use of the autoclave test for determining the tendency of glazed clayware to craze in service. Based on a study of the effect of time and pressure on the test results, the author recommends steaming for two hours if the pressures are less than 100 lb. per sq. in. and for one hour if greater pressures are used. Four tests of one hour each were found to be more severe than one test of four hours duration.

(263) L. A. Palmer and J. V. Hall, "Durability and Strength of Bond Between Mortar and Brick." U. S. Bureau of Standards *Journal of Research*, Vol. 6, No. 3, p. 473 (1931).

Specimens composed of two bricks bonded together with either a 1:3 portland cement or a 1:1:6 cement-line mortar were tested for strength or durability of bond. Neither the brick nor the mortar was disintegrated by 50 freezings and thawings, but several bond failures resulted. The wetting of rapidly absorbing brick before making the joint improved the bond. Compressive loads on joints greatly reduced the tendency to failure during freezing tests.

# DISCUSSION ON WEATHERING CHARACTERISTICS OF MASONRY MATERIALS

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MR. R. S. GREENMAN[1] (*presented in written form*).—In studying the various papers submitted in the "Symposium on Weathering Characteristics of Masonry Materials" I have been impressed by the clear and understanding presentations of the various papers by their respective authors.

In the paper by Mr. Viens on "The Performance of Concrete in Exposed Structures" I noticed in his "Conclusion" this sentence: "There are a number of men scattered across the country today who have devoted much of their lives to a study of concrete and they are pretty well agreed that where the material is handled in a satisfactory manner, its performance comes up to expectations."

This statement—together with the last statement in his "Conclusion"—are ones in which I heartily agree. I have long felt that *good* concrete—rightly made—is a lasting and permanent concrete. Of course, proper materials are necessary but there have been instances where exposed structures which contain what might generally be considered to be inferior materials have yet shown long life. I am not advising such practice in the selection of material but I believe too much stress is being laid at present on the question of aggregates over and above that of workmanship; for example, in the present case of certain state concrete highways. Aggregates should be good but even good aggregates do not guarantee a durable and weatherable concrete.

It was in a paper on "Examination of Concrete Failures for Their Determining Causes," presented before this Society in 1914,[2] that I made this statement: "The reasons for poor concrete have been proportioned as being 90 per cent due to poor workmanship, 8 per cent due to poor aggregates, and 2 per cent to poor cement." This statement was an opinion which apparently has been quite generally accepted as near the truth.

Now then, the suggestion by Mr. Viens that cements and aggregates must be supplemented by proper handling and workmanship is emphasized by him when he suggests that the large percentage of concrete is permanent concrete and when he brought out that, even where the mass of concrete is good, there may be some small portions of a structure which show disintegrations. With these conditions in mind, can it not be stated that, if the same materials are used throughout a structure, any portion failing must fail because of some other primary cause; and that cause must be handling, manipulation, or curing?

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[1] Consulting Engineer, Albany, N. Y.

[2] R. S. Greenman, "Examination of Concrete Failures for Their Determining Causes," *Proceedings*, Am. Soc. Testing Mats., Vol. XIV, Part II, p. 321 (1914).

This may be illustrated by references to two types of structures. Take for instance a long stretch of concrete pavement. There are many slabs and panels which are giving good service and then there may be in the same road an occasional slab, or perhaps the ends of adjoining slabs, which show a disintegration—possibly a scale or a ravelling—but, as Mr. Viens has suggested, these bad portions or sections may be only a small percentage of the whole. What then must be considered as the primary cause for this failing portion, materials or workmanship?

And, again, let us refer to a dam which has been much examined and written about. This dam was built in that period when extremely wet mixes were being used. In it there was used a coarse aggregate which was from a limestone rock having thin seams of clay. This rock was a Trenton limestone having characteristics similar to those of a rock mentioned in the paper by Mr. Hanson. This dam, too, was largely of cyclopean masonry and large blocks of stone were placed as "plums." Horizontal seams were prominent in the structure; and, although "keys" had been used to break seams or horizontal joints, considerable water seeped through to the down-stream face of the dam and quite an extensive scale of calcium carbonate was there formed. As the water continued to seep through, it was held back of this scale, absorbed by the concrete, and then a surface disintegration began. Where no seepage took place and no consequent scale formed, there was but little evidence of disintegration. This was particularly so on the up-stream face. Now had a good, dense concrete with a right water-cement ratio and with consistent attention to details of placing been used, one would not have found this concrete to have so badly disintegrated—notwithstanding the weakness in the aggregate due to the clay seams of the rock.

This statement has been made thus positively because there have been several recently published statements by men nationally known in the concrete field who hold the same view as expressed above—namely, that good concrete is often made from aggregate not entirely perfect. And referring to the paper by Mr. Jackson and under his discussion of "Freezing-and-Thawing Tests" one will find some endorsement of this opinion regarding the effect of this kind of aggregate. Again I would state that I am not advocating the use of inferior materials but I would emphasize the thought that close attention to construction detail must be given, even with entirely satisfactory materials, if permanent and durable concrete is to be secured.

Another thought expressed by Mr. Viens deserves consideration and commendation: A prevalent tendency to consider the design of a concrete too largely for its strength quality alone. We know that a dense, impermeable concrete resists weather conditions and it is a well-known fact that strength may be secured even with density not a prominent characteristic of the concrete; why then should not both qualities be essential parts of the concrete? But density is not only a factor of design, as is strength, but even more so it is a product of manipulation and curing.

The statement by Mr. Hanson in his paper on "Weathering of Aggregate" that "No relation was found to exist between the durability of the rock and any of its absorptive properties" may at first appear to disagree with the above expressed opinion regarding density of concrete being a factor of its durability against influences of weathering. In Table IV of his paper he gives a maximum absorption of 4.18 per cent (excepting, of course, the sandstone) and one can readily agree that little rock, with absorptive qualities as his table indicates, disintegrates by weathering through frost action alone but rather from the releasing of the binding material through normal weathering reactions. Concrete of similar low absorptive qualities would also seldom disintegrate because of weathering if its "cement-paste" furnishes a binder of a standard normal quality.

Mr. Kessler in his paper on "Weathering Test Procedures for Stone" begins by stating that "In this country frost has been proclaimed the chief weathering agent . . ." And then, too, he states: "The interesting conclusion of one experimenter was that it would require 10,000 years to weather one inch from the exposed face of marble." It may be considered idiotic to compare normal concretes with marbles but nevertheless it can be held that there is a relationship in the durability of the two based on the relative density and absorptive qualities.

With all the care that can be taken in the design of the mixture and the selection of materials for a concrete, it can be nullified if proper and consistent workmanship is not used in making up the concrete structure. This may seem trite. However, we are considering the weathering of materials. Where intelligent precautions have been taken in the making of the concrete, investigators of it for characteristics generally find that concrete seldom fails through normal weathering conditions and withstands many natural cycles of freezing and thawing.

In conclusion I would again quote Mr. Viens—"What is possibly most urgent today is the necessity that the knowledge which has been gained by the few be spread to others where it can be applied more generally; and particularly must those in ultimate charge of concrete work be impressed with their responsibility for the application of the better methods."

MR. H. S. BRIGHTLY.[1]—In respect to one point brought out in the paper by Mr. Hanson, the apparent inability to correlate "absorption" with resistance to damage from freezing. I should like to mention that the investigations of the weathering resistance of building stones have developed results indicating that the size and general character of voids has a decided effect. In other words, a rock with very fine, that is, small microscopic voids evenly distributed through the rock, having similar absorption to another rock in which the voids are very much larger, and of a more cellular nature, that is, sealed or closed in a sort of crystalline enclosure, of dense formation: that while the two stones may have a similar void space and therefore relatively similar absorption, the one with the very fine voids will show an entirely different resistance to freezing-and-thawing tests than the one with larger cellular voids.

I think that both Mr. Loughlin, of the U. S. Geological Survey, and Mr. Kessler, of the Bureau of Standards, have by tests developed information something along that line. I believe that Mr. Loughlin in one of his reports pointed out especially that the size of the voids and the general character of the rock formation has a material effect on the resistance to freezing and thawing.

MR. F. O. ANDEREGG.[2]—Referring to the paper by Mr. Kessler, the reactions of both freezing water and sodium sulfate solutions are dependent in part on the oriented growth of crystals. Ordinarily one thinks of the volume of expansion of ice as giving the bulk of the disintegrating reaction. One need only recall the different effect, for instance, of the rapid freezing of meat, as compared with very slow freezing, on the texture of the resulting product caused by, in the one case, a multitude of fine crystals, and in the other case a smaller number of large crystals which latter have produced disintegrating pressure on the meat structure. The same would apply if we were considering building materials. We have a variety of results of the sodium sulfate test carried out by different people under different conditions. The rate at which that solution cools is going to affect the growth of those crystals to a very considerable extent, and by taking advantage of this fact and introducing a very slow rate of temperature lowering, say one degree per hour or still lower, it is possible to produce tremendous pressures within the material in a comparatively short time.

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[1] Secretary-Director, Building Stone Association of Indiana, Inc., Bloomington, Ind.

[2] Consulting Specialist on Building Materials, Pittsburgh, Pa.

In addition to this, of course, sodium sulfate has some chemical reaction, especially in contact with cement products. To minimize these, it is desirable to maintain the contact for as short a period as possible, and this can be done by a slow cooling rate. In this way I have obtained far more disintegration of concrete in 15 cycles than in 60 cycles of freezing and thawing. In the one case there was complete disintegration; in the other case there was 5 per cent disintegration.

The method spoken of by Mr. Kessler, of adding a little sulfuric acid before making tests, is the logical one to follow, because in all building materials you often have accumulation of considerable amount of sulfate. The crystal pressures arising from low rates of cooling are superimposed upon the effects due to freezing alone, and, as he showed, the results are much more apt to be in accordance with the actual weather conditions.

MR. BRIGHTLY.—I wish to emphasize one point brought out in Mr. Kessler's paper, that the salt crystallization test, however useful, is not a dependable forecast of the weathering or weather-resistance of building stone, and that under certain conditions, certain stones subjected to that test will disintegrate rapidly which really have superior weathering qualities when placed in the building under ordinary conditions, and for that reason this form of accelerated test is not a thoroughly reliable criterion of the relative weathering properties of different building stones. That is, up to the present time we have been unable to correlate the salt crystallization test with freezing under actual conditions of exposure.

MR. D. W. KESSLER.[1]—As pointed out in this paper various people have tried to establish a relation between the sodium sulfate test and the freezing test for materials used strictly as building stone and very few of them have ever obtained anything very encouraging. I think it is quite true that different materials show different resistances and different ratios of resistance. Some are more resistant to the sodium sulfate test than to actual freezing while with others it is just the reverse. That conclusion also seems to be true of other salts. A great many salts have been tried in solution to bring about a crystal pressure as just mentioned by Mr. Anderegg. They usually show different degrees of severity, and as far as I know none of them will correlate satisfactorily with freezing tests.

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[1] Civil Engineer, U. S. Bureau of Standards, Washington, D. C.

The British Research Board gives a good deal of importance to the use of the crystallization test, but I do not think they do it to determine frost resistance as much as to determine the effect of the crystallizing salts in the materials themselves. In Great Britain a great many of the buildings are of limestone. Large amounts of soft coal are used there so that the action of sulfur acids from its combustion produces gypsum which forms beneath the surface and crystallizes, or else is produced on the surface and is carried into the pores by rain water thus producing the same type of action.

# MODERN PAVING EMULSIONS, TYPES, CHARACTERISTICS AND TEST METHODS

By C. L. McKesson[1]

## Synopsis

Emulsified asphalt, for many years used almost entirely in cold asphaltic mixtures prepared in central mixing plants or by roadside mixing, is now coming into general use for road mix and penetration types of construction work.

Two distinct types of paving emulsion are now being extensively produced, designated according to their intended use as "mixing emulsion" and "penetration emulsion." The composition and physical characteristics of these products differ greatly and the materials are not interchangeable in use.

Mixing emulsion must be sufficiently stabilized in the process of manufacture to render it miscible in water and to make it sufficiently "slow setting" to permit it to be mixed with aggregate prior to the coalescence of the asphalt in the emulsion. Asphalt content affects viscosity, and asphalt content and viscosity vary according to the mixing method to be used.

Penetration emulsions have as a primary requisite "quick setting" property. That is, the asphalt in the emulsion must coalesce in the least possible time after application to prevent damage from rainfall during construction, to permit successive construction operations to proceed in quick succession, and to prevent wastage of cover stone chips under traffic after work is completed. The "quick setting" property requires the production of an emulsion containing little or no emulsifying agent.

It is also necessary that penetration emulsion remain uniformly dispersed during shipment and in storage prior to use.

Necessary tests for "mixing emulsions" include miscibility, stone coating, setting time (the washing test and the Myers' test being proposed), and tests for amount and character of residue.

Necessary tests for "penetration emulsion" include tests for setting time (the washing test or Myers' test being proposed), settlement, viscosity, amount and character of residue.

This paper discusses the required properties of the emulsion and the test methods believed to determine the suitability of the product for its intended use.

Emulsified asphalt has been used in the United States in road maintenance and construction work over a period of about twenty years. Up to 1927, emulsions were generally used in cold mixtures, prepared in roadside mixing plants, or mixed by hand. The term "emulsified asphalt" was understood by highway engineers and testing engineers to refer to emulsions which were miscible in water

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[1] Director of Engineering and Research, American Bitumuls Co., San Francisco, Calif.

and which were suitable for such work. The emulsions were manufactured with soap stabilization, containing 2 to 8 per cent of soapy material, or were made with colloidal clay as the emulsifying agent. The only test methods in use for determining the properties of emulsions were the miscibility test, stone-coating test, and distillation method for obtaining residue in accordance with the Society's Tentative Methods of Testing Bituminous Emulsions (D 244 - 28 T).[1]

In 1927, "quick-setting" penetration emulsions made their advent in America, coming from Europe where their use had become quite extensive. These products differed radically in composition and in physical characteristics from the earlier emulsions, and inasmuch as the Society's tentative test methods had been built up around previous mixing emulsions it has been necessary to develop new tests to differentiate between suitable and unsuitable products of the new types. The use of the quick-setting penetration emulsion has increased rapidly, and it is reported that more than twenty million gallons of such emulsions have been used during the current year in the United States. Several important state highway departments have adopted penetration emulsion as a standard paving material and the use of such products is now common in at least forty states.

## TYPES OF EMULSIONS

Emulsions are now generally classified in two types—mixing emulsions and penetration emulsions—and have the following characteristics.

*Mixing Emulsion.*—Mixing emulsion must be capable of dilution and must coat aggregate by machine or hand mixing methods. It must be sufficiently slow "setting" to permit of uniform distribution of asphalt in the coatings, and of handling and manipulation of the mixture incident to construction.

The asphalt content in emulsion for use in machine or hand mix is usually 65 to 70 per cent, viscosity being of minor importance. In road-mix emulsions, low viscosity is required to permit of application with truck distributors, and the asphalt content is usually between 55 and 60 per cent.

*Penetration Emulsion.*—Penetration emulsion has physical characteristics peculiar to the special use for which it is intended. After application, such emulsion is ineffective as a binder until the dispersed particles of asphalt coalesce or demulsify and form a coating of asphalt on the metal in the pavement.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 952 (1928); also 1930 Book of A.S.T.M. Tentative Standards, p. 455.

The time required for coalescence is usually referred to by construction engineers as the "setting" time or the "breaking" time of the emulsion. In penetration construction work, rolling cannot safely proceed while the emulsion is setting because at this stage the asphalt is readily stripped from the stone by movement under the roller. Successive applications which are used to build up coatings of asphalt on the stone are not fully effective if the previous application of emulsion has not set. Stone chips and keystone are not bound in place by asphalt until the emulsion has coalesced and adhered to the stone.

Rainfall prior to setting or demulsification tends to remove emulsion from the stone. Difficulty has been experienced in penetration construction work where stabilized, slow-breaking emulsions have been used, due to damp weather, rainfall or the inability to protect the aggregate from traffic during the long period following the spreading of the emulsion and prior to its permanent coalescence. It is believed that the quick setting or quick breaking property is the most important single characteristic of a penetration emulsion and one which must always be given consideration in selecting emulsion for construction work of this type.

A penetration emulsion must be sufficiently stable to prevent coalescence and formation of slugs or coagulated particles which will interfere with proper application through pressure distributors, and must also remain in a uniform state of suspension, free from sedimentation, during shipment and in storage until required for use. With the old type of mixing emulsion, freedom from settlement was not an important consideration because these mixing emulsions were shipped in drums or barrels, and were remixed prior to their use. They were seldom stored in tanks or in tank cars for long periods of time.

The asphaltic content in a penetration emulsion is governed by the required viscosity and by the thickness of coating required on the stone. In deep penetration work and where there are several applications of emulsion, a 50 to 55-per-cent emulsion has been found to give satisfactory results. In shallow surfacings and where only one or two applications are used, higher viscosity is permissible and 55 to 60-per-cent asphaltic content seems most suitable.

## TEST METHODS

Having two very different types of emulsions not interchangeable in use, it becomes necessary to consider test methods which will differentiate between the two types and which will also make possible the elimination of unsatisfactory materials in each type. As might be expected, some test methods are applicable to one type only.

The discussion will first be directed to tests for determining physical properties of penetration emulsion, then mixing emulsion and, last, to tests for characteristics common to both types.

*Tests for Penetration Emulsion:*

"Quick setting" is essential in penetration emulsion, and "slow setting" is equally necessary in mixing emulsions. "Setting time" is therefore fully as significant and important in connection with emulsion as it is with portland cement.

TABLE I.—RESULTS OF TESTS ON VARIOUS BITUMINOUS EMULSIONS.

| Asphalt Residue in Emulsion, per cent | Saponifiable Oils, per cent | Percentage Demulsification, Myers Test (for Penetration Emulsion), 35 cc., 0.02 $N$ $CaCl_2$ | | | Asphalt Remaining on Stone, Washing Test, per cent | | Percentage Demulsification, Myers Test (for Mixing Emulsion), 50 cc. of 0.1 $N$ $CaCl_2$ | Proposed Classification of Emulsion |
|---|---|---|---|---|---|---|---|---|
| | | Oakland Laboratory[a] | Baltimore Laboratory[b] | Sacramento Laboratory[c] | Crushed Stone | Gravel | | |
| 51.2 | 0.00 | 100.0 | 97.1 | 93.0 | 88 | 96 | 100 | Quick-setting penetration emulsion |
| 52.2 | 0.25 | 38.3 | 36.4 | 32.4 | 65 | .. | ... | Slow-setting penetration emulsion |
| 52.6 | 0.50 | 20.6 | 14.8 | 14.2 | 38 | .. | 100 | Slow-setting penetration emulsion |
| 53.4 | 0.75 | 10.8 | 8.1 | 10.8 | .. | .. | ... | Not suitable for mixing or penetration work |
| 53.8 | 1.00 | 5.5 | 4.1 | 7.0 | 37 | 56.3 | 96.6 | Not suitable for either type of work |
| 54.6 | 1.25 | 3.6 | 1.3 | 5.2 | .. | .. | ... | Not suitable for either type of work |
| 55.2 | 1.50 | 2.4 | 1.8 | 3.0 | 21 | 32.9 | 71.8 | Quick-setting mixing emulsion |
| 56.4 | 2.00 | 1.1 | 1.4 | 2.2 | .. | 14 | 58.3 | Quick-setting mixing emulsion |
| | 2.50 | ... | ... | ... | .. | 5 | 46.3 | Slow-setting mixing emulsion |
| | 3.00 | ... | ... | ... | .. | 2 | 40.3 | Slow-setting mixing emulsion |

[a] Tests made at American Bitumuls Co. Laboratory at Oakland, Calif. (Oven evaporation of residue).
[b] Tests made at American Bitumuls Co. Laboratory at Baltimore, Md. (Oven evaporation of residue).
[c] Tests made at California State Highway Laboratory, Sacramento, Calif. (Hot Plate evaporation of residue).

*Washing Test.*—Following the theory that the degree to which an emulsion had set or coalesced in a given period of time could be measured by the amount of emulsion which could be washed from the stone at the expiration of that time, the author, in 1928, proposed a "washing test" made in the following manner:

Clean gravel of uniform quality from a source which permitted of duplication for future test work was selected. Two 1000-g. samples passing a ½-in. and retained on a ¼-in. screen were placed in wire baskets and dried to a constant weight in an oven at 125° F. The baskets were immersed for two minutes in the emulsion to be tested. After removal from the emulsion, one basket was allowed to drain exactly 30 minutes and was then washed with tap water for 15 minutes, after which it was dried to a constant weight in a suitable container in an oven at 125° F. The increased weight due to the deposited asphalt was noted. The other basket was dried to constant weight in the same manner without spraying with water. The asphalt remaining on the aggregate in the washed basket was calculated as a percentage of the asphalt deposited on the aggregate in the unwashed basket.

Hundreds of tests have been made by this method in routine research work and valuable data obtained. It is believed that this test approximates the condition which occurs in the field when there is rainfall soon after the application of emulsion and that it therefore reflects an actual service condition. In order to secure uniform results, it was necessary to further standardize some of the details specified in the test method, as follows:

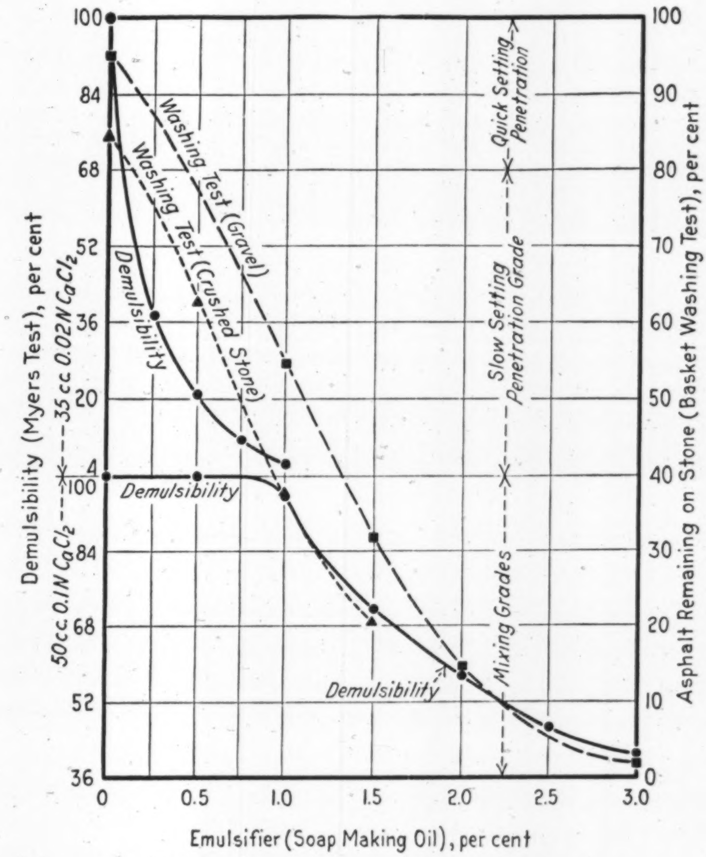

Fig. 1.—Results of Tests on Bituminous Emulsions.

1. The temperature of rock and emulsion must be maintained at 25° C. (77° F.) during the test.

2. Clean rounded river gravel graded within limits shown must be used.

3. The test must be made in a room or compartment maintained at 25° C. (77° F.) and at a relative humidity of about 50.

4. The coated aggregate must be washed with a stream through a $\frac{1}{4}$-in. hose at the rate one gallon per minute until the wash water is clean.

Typical results obtained by this test method are shown in Table I with the results of other tests. The results are also shown on Fig. 1.

Quick-setting emulsion suitable for penetration work shows 80 to 100 per cent of the asphalt remaining on the stone after washing in 30 minutes. "Slow setting" emulsions suitable for mixing show 2 to 40 per cent of the asphalt remaining. It will be seen in Fig. 1, that there is a well-defined relation between setting time as determined by this test and the percentage of emulsifying agent used.

*Myers Demulsification Test (For Penetration Emulsion).*—Mr. Joseph Myers, Chief Chemist, of the New York State Highway Department, suggested a test method which appears to give quickly, simply and accurately, the measure of demulsibility of an emulsion. The method as proposed by Mr. Myers, and slightly modified in the Laboratories of the American Bitumuls Co., with Mr. Myers approval, is as follows:

> One hundred grams of emulsified asphalt is placed in a tared 600-cc. glass beaker and 35 cc. of 0.02 normal solution of calcium chloride is added during a period of approximately two minutes, being continually stirred with a glass rod. The contents of the beaker are then drained through a No. 14 sieve and the unbroken emulsified asphalt in the beaker and on the rod is rinsed through the sieve with distilled water until there is no appreciable discoloration of the rinsing water. The beaker, rod and sieve are then dried in an oven at 163° C. (325° F.) for two hours and weighed. The percentage of asphalt residue deposited in the beaker and on the rod and sieve are taken to be the difference between the weight of the beaker, rod and sieve after drying and their tared weight determined at the beginning of the test.

This test is evidently founded on the tendency of $CaCl_2$ to reverse an oil in water emulsion. As a result of stirring while the calcium chloride solution is being added, a clot of coagulated asphaltic residue is formed, varying in size with the setting time of the emulsion. Subsequent washing with distilled water removes the separated water of emulsion and unbroken emulsion. Afterward, the water in the clot is removed by evaporation at 325° F., leaving the asphaltic residue. The ratio of the residue so obtained to residue in the emulsion prior to testing is taken as the index of the demulsibility of the emulsion. A large number of experiments have indicated that a quick-setting penetration emulsion will show a residue, in this test, of 80 to 100 per cent of the total residue contained in the emulsion.

Table I and the solid line curve in the upper half of Fig. 1 show the comparative results of the Myers demulsibility test made as described above, and the washing test on samples ranging from pure unstabilized penetration emulsion to extremely slow-breaking mixing emulsions containing a strong sodium soap in the adsorbed film.

Results are included of tests made at the American Bitumuls Co. Oakland laboratory, at American Bitumuls Co. laboratory at Baltimore and on identical samples at the California State Highway Department laboratory (residue obtained by evaporating sample on electric hot plate). It will be seen from this table that the test produces reasonably uniform results in the hands of different operators in widely separated laboratories.

The present indications are that the Myers test will give as accurate an index to the relative demulsibility of emulsions as the washing test, and that it is much simpler. It is possible that the presence of certain emulsifying agents may not, however, be detected by the Myers test. In such cases the lowered demulsibility can always be measured by the washing test.

It has been found that storage of samples in tinned containers tends to lower the demulsibility of the emulsion. Samples should be stored in iron or glass containers.

*Settlement Test.*—Where emulsions are shipped in tank cars or are stored for any appreciable time after manufacture, the tendency toward sedimentation becomes a serious consideration, even if the emulsion is stable and does not tend to coagulate. In order to measure this tendency toward sedimentation, the Research Department of the American Bitumuls Co. in 1928 developed a test which has been continuously used in the laboratories of the company since that time and which has been adopted as a standard test by many state highway departments and other important users. This test has become known as the "settlement test" and is described as follows:

> Two samples of 500 cc. each from the average sample are placed in air-tight, covered cylindrical glass containers with an inside diameter of 4 to 5 cm. and allowed to stand for ten days at laboratory atmospheric temperature. Fifty grams are then siphoned from the top of each sample, care being taken not to disturb the remainder of the content. Approximately 400 cc. of the emulsion are then removed from each sample with the siphon and finally a 50-g. sample is taken from the bottom of each container. The samples from the top and from the bottom of the container are then tested in accordance with the test for "loss at 163° C." The variation between the average amount of residue of top and bottom samples shall not be more than 6 per cent.

This settlement test gives an accurate measure of the tendency toward sedimentation regardless of coagulation and is believed to be indispensable in any specification where uniformity of asphalt coating on aggregate is desired. Many attempts have been made to develop a substitute test by centrifuging or in some other way which would eliminate the ten-day interval, but so far no substitute test has been developed.

It has been found entirely feasible to produce emulsions with either Mexican or California asphalt which will meet the test requirement above set forth.

*Tests for Mixing Emulsions:*

*Miscibility; Stone Coating; Myers Test.*—Test methods applicable to mixing emulsion but not to penetration emulsion are:

1. *Miscibility* } A.S.T.M. Tentative Method of Testing
2. *Stone Coating* } Bituminous Emulsions (D 244 - 28 T)
3. *Myers Test (for Mixing Emulsion)*

Slow-setting properties essential in a mixing emulsion are partially assured by the miscibility and stone coating tests. It is desirable, however, to be able, for many uses, to determine the relative setting time or demulsibility of mixing emulsions.

The Myers test for penetration emulsions (using 35 cc. 0.02$N$ $CaCl_2$) as will be noted in Table I, shows relatively little difference between emulsions having 0.75 to 2.0 per cent of soapy emulsifying agent. To differentiate accurately between various grades of mixing emulsions, the Myers test was modified by using 50 cc. 0.1 $N$ $CaCl_2$, the test method otherwise remaining unchanged.

The results of this test are shown in Table I and by the solid line curve in the lower half of Fig. 1. The relatively demulsibility is brought out clearly and corresponds with the performance of the materials in mix work. Demulsibility of less than 70 per cent by this modified method is necessary to permit of mixing with coarsely graded aggregate and for some purposes a much lower percentage demulsible is required.

*Washing Test.*—The washing test is also applicable to mixing emulsions. Table I and Fig. 1 indicate that not more than 30 to 40 per cent of the asphalt should remain on the stone after washing if the material is sufficiently stabilized to be suitable for mixing purposes.

*Settlement Test.*—The settlement test becomes applicable to mixing emulsions which are to be stored in tanks or shipped in tank cars. Where material is to be stored in drums, sedimentation becomes more or less unimportant because the material can be stirred or agitated and remixed prior to use.

*Tests for All Types of Emulsion:*

Tests for amount and quality of residue are common to all types.

*Distillation Test.*—The distillation test made in accordance with the Society's Tentative Methods of Testing Bituminous Emulsions (D 244 - 28 T) is frequently used to determine amount of residue and penetration. Solubility and ash tests are made on the residue. There are two very serious objections to the distillation method. First, it is slow, tedious and requires a high degree of skill. Second, it greatly lowers the penetration of the asphalt recovered as residue. Frequently the drop in penetration is 50 to 100 deg.

*Evaporation Test.*—Many large users have specified an open cup or constant-temperature oven evaporation method, using samples of 50 to 200 g. The samples are evaporated at 163° C. (325° F.) for 2 to 3 hours. These tests give percentage of residue as accurately as the distillation test and are less difficult to make. They tend, however, to lower the penetration of the asphalt in the residue, although to a less extent than does the distillation method.

A very simple test for amount of residue where no further tests are required is to place a 20-g. sample on a tin lid and evaporate on an electric plate at about 163° C. until oil vapor begins to come off. This usually requires about eight minutes. This test has been found to be as accurate in determination of quantity of residue as the distillation test.

Mr. H. G. Bennett, asphalt testing engineer in the laboratory of the California Highway Department, devised a method of recovering asphalt from emulsion for penetration tests by precipitating the asphaltic content with acid, neutralizing and squeezing the water out of the asphaltic residue by pressing through a set of rolls. The asphalt recovered was found to have the same penetration as when emulsified.

More work must be done to perfect some simple test method for recovering asphalt in emulsion without altering its physical characteristics.

*Viscosity.*—Viscosity is largely dependent on asphaltic content and tests for viscosity are ordinarily not required. When they are to be made on emulsions containing 60-per-cent or less asphaltic content, satisfactory results are obtained by the use of the Saybolt-Furol viscosimeter using 60 cc., 25° C. (77° F.). The Engler viscosimeter is unsatisfactory for testing emulsions. No satisfactory viscosity test has been devised for emulsions having 65 to 70-per-cent asphaltic content.

## CONCLUSIONS

The following conclusions are offered:

1. Tests for penetration emulsion should include:
   (*a*) Washing test, or Myers demulsibility test (35 cc. 0.02 *N* $CaCl_2$),
   (*b*) Settlement test, and
   (*c*) Residue Test.
2. Tests for mixing emulsions should include:
   (*a*) Miscibility Test,
   (*b*) Stone coating,
   (*c*) Washing test, or Myers test (50 cc. 0.1 *N* $CaCl_2$),
   (*d*) Settlement test (for emulsion to be stored), and
   (*e*) Residue test.

# DISCUSSION

MR. J. S. MILLER, JR.[1] (*presented in written form*).—Mr. McKesson is to be thanked by the industry for his able presentation of the subject of emulsions of asphalts and the assembly of the collective data upon them and their uses in highway work.

In my twenty odd years of association with highway problems I have always noted the tendency of highway engineers to try new methods of construction and how there seem to be cycles wherein previously tried materials or methods are revived. I bring this up merely to indicate that tendency without in any way reflecting upon the suitability of emulsions for construction but rather to sound a note of caution in expecting emulsions to be the one best method or material. Emulsions, I believe, have their limitations. By this I mean an emulsion satisfactory for mixing methods with a given type or grading of stone aggregate may prove utterly unsuited to a different grading of that aggregate or to another aggregate. Up to the present, we have been unable to classify this behavior definitely. Trap rock and slags appear to be the worst actors while dirty aggregates are always troublesome. I am discussing those types of emulsions dependent upon apparent solution of the emulsifying agent rather than an insoluble inorganic type of emulsifier such as colloidal clay.

Referring to Mr. McKesson's paper, I cannot subscribe to the general statement of "viscosity being of minor importance," when referring to those intended for machine or hand mix, while for truck distribution emulsions of low viscosity (thin) are now required. Viscosity determines the thickness of film remaining upon the aggregate. One of the chief causes for failure when using emulsions is in having too thin a film upon the aggregate. I firmly believe we shall find emulsions intended for penetration construction tending towards the thicker viscosities to provide a thicker film and distributors will be modified so as to handle them.

How much has our notion about amount of emulsion necessary for a penetration job been influenced by European practice? Did not the exigency of the situation and economic considerations govern rather than sound deductions from experience gained with hot applied binders? Tar as a binder is relatively cheaper than asphalt over there. To increase the cost of the asphalt binder by emulsifying it, might

[1] Director, Technical Bureau, The Barber Asphalt Co., Maurer, N. J.

have put it out of the running from a cost standpoint so the next best thing was to reduce the amount used. I do not believe any less asphalt can be used per square yard of pavement for a given thickness of wearing surface simply because it is applied as an emulsion. Then, too, a so-called "semi-grout" method was developed where the bulk of the thickness of the wearing surface was filled so that the emulsion could not go deeper into the wearing surface than the upper void space so provided would permit. This results in a stone dust-bound wearing surface of, say, three inches thick with the top one inch to three-quarters inch only bound with asphalt. We would never consider such construction for hot penetration satisfactory. I am not belittling hot-penetration construction either for I firmly believe in hot-penetration work.

Consider now what happens when a quick break emulsion breaks on an aggregate. If by break is meant a skinning over at the surface then the emulsion film beneath this skin and in contact with the aggregate remains either as an emulsion or if it breaks the water must be trapped in by the outer film.

If the emulsion breaks at the interface between it and the aggregate what happens to the water thus set free? Does it dilute the rest of the emulsion of the film and does it by such dilution cause breaking of the balance of the film? Perhaps there may be a uniform breaking throughout the film as water evaporates from the surface and disturbs the equilibrium of the system.

Then, too, we have the more severe action of some aggregate over others that may hasten this so-called "setting" or "breaking" time.

Whatever the mechanism, I have discussed it in this manner so that others using emulsions for the first time will not expect certain definite results from an emulsion because by the Myers demulsification test the emulsion showed a certain figure. As a laboratory tool this method will certainly give comparisons between one emulsion and another, but until thoroughly studied for a given type of emulsion for a given type of aggregate, one cannot predict just what will happen in the field.

Recently I received a report from one of our field engineers on a comparative application of two different emulsions in surface treating in the same city. These emulsions differed as to Myers demulsification figure, but the quicker breaking type showed a slower set-up by several hours than the more stable emulsion. This may have been due entirely to the character of road surface upon which it was applied.

In our experience it has been found necessary for any supposedly standard aggregate for either the washing test or the so-called "stone test" to be absolutely clean, preferably by washing the aggregate. We have not used this test and cannot therefore say very much about it, but it would seem necessary to specify how close the hose should be held to the aggregate so no difference in force will be used between one test and another.

We have done considerable work on the Myers demulsification test and other methods and have the following suggestions to make. The test, during the mixing of the reagent and the emulsion must be run at a very definite temperature and we are using 77° F. Also a brass or bronze wire No. 14 sieve behaves entirely differently and we would suggest an iron wire sieve for this purpose. Sieves of brass wire seem to be effective in causing additional breakdown of the emulsion while iron wire sieves do not apparently affect the result.

Care must be exercised in considering the meaning of the settlement test. Settling or sedimentation may occur in any type of emulsion, so may creaming. If by settling or sedimentation is meant breaking out of asphalt particles that is one thing, whereas, creaming is another phenomenon, is not associated with breaking and may be either a rising or falling cream. Creaming is probably a function of variable particle size and ratio of specific gravity of the dispersed particle to that of the continuous phase.

The question is raised as to whether the tentative method of this Society does in the distillation test give a residue equivalent to the original consistency. The section of the Society's Committee D-4 on Road and Paving Materials, which carried on the work leading to this tentative method considered and tried open-pan evaporation, separation of the asphalt by the use of alcohol, acid or other reagents and concluded the distillation test gave an indication of what the hardness of the asphalt left in use would be, and specification ranges of 75 to 100 points in penetration seem to be satisfactory for most uses while for certain specialized uses this range has been narrowed but in no case are we expecting ranges to be maintained or required as close as in hot work. Any method other than distillation will not catch other volatile materials than water which might be present.

Viscosity may be dependent upon asphalt content, but there are many other and we believe more important factors. Entrained air, character of emulsifier and particle size are all factors. Emulsions tend to be thisotropic and assume gel-like structures or a false body, hence any type of outflow viscosity instrument is not applicable; in fact a true viscosity of a gel could not be determined. The MacMichael instrument however does give reliable information after breakdown of the gel structure, if formed, and has the advantage of doing the necessary work to break such structures in most cases.

In conclusion, I want to say that I believe emulsions will play an important rôle in repairs and maintenance; under ideal conditions they may become popular for construction of wearing surfaces if their cost is justified *versus* hot mixing or hot penetration, and our entire idea is one of constructive criticism, hoping thereby to warn engineers not to approach emulsion use as a positive cure-all but realize each situation requires careful study of the local conditions, including the economics of the problem.

MR. C. L. MCKESSON.[1]—Mr. Miller discusses specification requirements, particularly with reference to the quantity of emulsion required in penetration work. He doubts whether the quantity of asphalt in penetration construction work can safely be reduced below that required in hot-penetration work, when the asphalt is applied in the form of an emulsion. This question is always uppermost in the mind of the engineer unfamiliar with emulsified asphalt penetration construction.

Quick-setting emulsions have now been in extensive use in penetration work throughout all of the important countries of Europe over a period of eight or ten years, and in the United States for more than four years. Many thousands of miles of such roads and pavements have been constructed and are now in service. It is the universal practice in this country and in the other countries to use an amount of emulsified asphalt equal to or even slightly less than the amount of hot asphalt which would have been used in the same type of work.

In the construction of the first piece of penetration pavement in the United States in which quick-setting emulsion was used, the asphalt content was varied on different sections in an attempt to check up on the European practice in which the amount of emulsion used equals the amount of hot asphalt previously specified. On this project, the sections in which the asphalt content was two-thirds that which would have been used in the hot form, give slight evidence of bleeding and displacement after four years of heavy arterial traffic. The sections where the total amount of emulsion (of 50 per cent asphaltic content) equalled the amount of hot asphalt ordinarily used are today in absolutely perfect condition without maintenance since completion in 1927.

Projects constructed with emulsion in which the asphalt content is approximately half that which would have been used by hot penetration methods are in service in almost every country and climate. Everywhere engineers familiar with this type of construction agree that this is sound practice, this decision being based upon observed results. The soundness of the present specification requirements has been universally demonstrated in actual practice.

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[1] Director of Engineering and Research, American Bitumuls Co., San Francisco, Calif.

In hot-penetration work in this country the practice has varied widely in the quantity of asphalt required. In warm sections the amount of asphalt used is generally about 60 or 70 per cent of that specified in the New England states. Even with this reduced quantity difficulty has been experienced due to bleeding and corrugating. In the Southwestern states, where long hot summers prevail, the amount of asphalt in hot-penetration work is usually about three-quarters gallon per square yard per inch of thickness. In the New England states a more or less uniform practice seems to call for one gallon per square yard per inch of thickness. The same general climatic considerations govern the use of emulsion—slightly lower quantities being used in hot climates. Other details of construction being properly attended to, it is safe and conservative to specify the number of gallons of emulsion (50 to 55 per cent residue) as would be otherwise specified in hot-penetration work. The use of additional quantities of emulsion above those actually required is detrimental in emulsified asphalt pavement exactly as it is in hot-penetration work.

MR. PRÉVOST HUBBARD.[1]—In connection with this question of the relative amounts of emulsion and of hot asphalt for penetration construction, I should like to ask Mr. McKesson if the size or grading of the aggregate is not an important consideration. That is, where a lower percentage of asphalt in emulsion may be used, is not the grading of the aggregate very much closer and denser than in the case of the hot-application asphalt?

MR. MCKESSON.—That is quite true. In the penetration type of construction work, instead of leaving the voids in the coarse aggregate open to secure penetration as is done with hot asphalt, the practice is to fill these voids as closely as possible first with clean, dustless stone chips, and then penetrate and coat the chips and the aggregate. Consequently, the finished pavement has a very much more complete grading from fine to coarse than would be or could be secured in any standard hot-penetration construction work. In hot-penetration work the voids between the coarser particles are bridged and filled to a considerable extent with pure asphalt. In emulsified asphalt penetration work these voids are filled with asphalt coated chips. A considerably less quantity of asphalt is therefore required, and a denser pavement obtained.

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[1] Chemical Engineer, The Asphalt Inst., New York City.

MR. HUBBARD.—The point I wanted to develop was that in penetration construction the difference in aggregate used for asphalt emulsion and for hot asphalt creates two distinct types of pavement which require different percentages of asphalt. If the character and grading of aggregate were the same, however, the proper quantity of asphalt to use would be the same irrespective of whether it was applied hot or in the form of an emulsion.

MR. MCKESSON.—On quite a large number of jobs we have recorded and checked very carefully the amount of aggregate used in any given thickness of pavement. We have found on an average of approximately 12.5 lb. of mineral aggregate, having 2.65 specific gravity, per inch of thickness per square foot. That corresponds in weight very closely to the average throughout the country for asphaltic concretes.

The difference in grading, of course, is that there is no fine sand in the emulsion type of penetration work, but large quantities of clean stone chips are used for filling purposes, and the voids are well filled with these chips and with the subsequent sizes of clean stone.

(*Author's closure by letter*).—Mr. Miller in his discussion brings up a number of interesting and pertinent questions touching on the characteristics of emulsions, and the test methods outlined.

I agree with Mr. Miller that a general statement to the effect that viscosity in emulsions is of minor importance would be quite incorrect. Mr. Miller evidently misinterpreted the paper in this regard. In the paper the statement is made—"asphalt content in emulsion for use in machine or hand mix is usually 65 to 70 per cent, viscosity being of minor importance." It is, however, pointed out that viscosity is important in connection with emulsion to be used in road mix work and applied with distributors. Emulsions with 65 to 70 per cent asphaltic content used in machine or hand mix invariably have a viscosity sufficiently high to insure an adequate thickness of coating—in fact, the viscosity of such products is frequently so high that dilution is necessary to accomplish proper mixing. In penetration emulsions, as is pointed out in the paper, the viscosity and asphalt content are both to be varied according to the thickness of coating required on the stone. The author believes that viscosity of all products for penetration or mix work which are to be applied with truck distributors can be determined with sufficient accuracy in the Saybolt-Furol viscosimeter. Emulsions to be used in plant mix due to high viscosity cannot be tested by this method and in such cases Mr. Miller's suggestion of the use of the MacMichael viscosimeter or some other similar instrument is quite useful. The old specification used for many years for this type of product—"that it shall be sufficiently fluid to permit of its being poured from a 2-in. bung of a drum"—is a rather crude requirement and some method should probably be adopted of measuring this characteristic of heavy emulsions. Mr. Miller properly points out that there are other conditions than asphaltic content which affect the viscosity of emulsions. This is particularly true of the heavy mixing type. The viscosity of penetration and road mix type of emulsions is not so appreciably affected by other influences than asphalt content. The very use to which they are put requires in any event they shall be sufficiently fluid to pass readily through a distributor.

Mr. Miller suggests that the Myers demulsification test should be run at a definite temperature if uniform results are to be obtained. Considerable work has been done in checking the effect of temperature and all of the reports which we have received indicate that temperature is not a critical factor. The low temperatures, such as, for example, 77° F. recommended by Mr. Miller, make the tests slow and more tedious, a considerable length of time being required to wash out all of the unbroken emulsion. Iron wire sieves have generally been used and should probably be specified in making the test a standard.

Mr. Miller comments on the report which he has received in which two emulsions differing in the Myers demulsification figure had been under observation and in which the quicker breaking type showed a slower set on the road than the more stable emulsion. Many factors enter into the time of set on a road project such as temperature, moisture on the road or the aggregate to which the emulsion is applied, humidity, amount of emulsion applied, etc. The Myers test does, however, give an accurate index of the relative breaking time of the emulsion and an emulsion which, in this test, shows a quick breaking time will generally show the same relative performance when applied to the road. In other words, while conditions may be so adverse that a quick breaking emulsion will require several hours for permanent coalescence and adhesion to the stone, a slow-setting emulsion, as determined by the Myers test, would require a much longer time under the identical conditions.

Mr. Miller opens an interesting discussion of the phenomena which occur when an emulsion "breaks" or "sets" with a quick breaking emulsion. A film of asphalt is first formed more or less completely surrounding the particle of aggregate by the quick coalescence of the asphalt contained in the emulsion. The water of emulsification is freed, escaping by evaporation, and in some cases by draining away from the coated aggregate. After the film is formed, it is not injured by subsequent dilution either by rainfall or by water in the emulsion. If allowed to remain undisturbed for a brief period the film adheres firmly to the aggregate and the asphalt becomes effective as a road binder. A slow breaking emulsion behaves quite differently. This type of emulsion being very stable, breaks either wholly or in part by evaporation and a water-resistant film is not quickly formed. During the setting period the film or coating formed with slow-setting emulsion is subject to dilution and serious damage if the pavement is subjected to rainfall or receives water from any source. In mixed work, slow breaking is necessary to accomplish proper coating and to permit of manipulation during the breaking period and in this process of mixing a considerable portion of the moisture content is undoubtedly discharged to the air. In penetration work, however, particularly in thick pavements, the coating on the stone dries very slowly and it is in the penetration type that quick setting emulsions are most indispensible. Regardless of the type which may be desired, as Mr. Miller points out, the Myers test is a laboratory tool which will give comparisons between different types of emulsion.

Mr. Miller very properly stresses the point that sedimentation may be either upward or downward—that is, that there may be a tendency of the bituminous material to collect either at the top or at the bottom of an emulsion. Obviously an emulsion in which the asphalt content is concentrated in either the top or bottom of a storage receptacle has not the homogeneity which will insure uniform application and uniform asphalt content or coating in the finished work. If the creaming is to the top, as pointed out by Mr. Miller, material drawn from the bottom of the storage tank would be deficient in asphalt and probably of low viscosity, while the material from the top of the tank would have a relatively high asphalt content and high viscosity. Good emulsions do not show appreciable creaming in either direction and the settlement test definitely measures this tendency, enabling accurate specification requirements which are preferable to the term sometimes used—"The asphalt in the emulsion shall remain uniformly dispersed until the emulsion is applied to the road."

Mr. Miller points out that the distillation test serves fairly well where ranges in penetration of 75 to 100 points are permitted. The author would urge, however, the substitution of a test method less tedious to make and which will produce as residue the asphalt in its original condition rather than a product which has been so greatly reduced in penetration by severe heating. Open evaporation gives much more quickly the percentage of residue and separation but the use of acid or other reagents furnishes residue similar to the asphalt used in the emulsion and to that deposited on the stone.

Mr. Miller properly points out in his discussion that emulsions are not fool proof. For that matter, no engineering material is fool proof. Care, intelligence and skill are necessary in handling emulsified asphalt products as in the uses of other materials. The rapidly growing use of emulsion throughout the world is encouraging in indicating that engineers generally are coming to understand the proper use of such materials and are achieving success.

# THE HIDING POWER OF WHITE PIGMENTS

By G. S. Haslam[1] and D. L. Gamble[1]

## Synopsis

This paper presents a comparison of results yielded by various hiding power methods, conforming in general to the tentative A.S.T.M. definition of hiding power, when the hiding powers of several white pigments were measured. An attempt is made to correlate the hiding power results with tinting strength values.

A comparison of hiding power tests employing a visual end point with those in which the photometric end point is used along with data obtained with a photo-electric cryptometer indicates that the eye is capable of detecting a closer contrast than 98 to 100, the assumption made in formulating the definition of hiding power. It is suggested that an attempt be made to definitely establish the correct contrast ratio.

Experimental results do not show a definite straight-line relationship between hiding power and tinting strength for the pigments measured. Theoretical consideration indicates that a straight-line relationship between these two properties of a pigment is not to be expected.

It has been found possible to determine hiding power with the same ease of manipulation as tinting strength and with as good or better reproducibility. It is suggested that hiding power should be measured and specified where hiding power is meant rather than tinting strength.

## Introduction

Considerable interest has been centered in the cooperative work on tinting strength and hiding power carried out during the past few years by Subcommittee VIII on Methods of Analysis of Paint Materials of the Society's Committee D-1 on Preservative Coatings for Structural Materials. Several new methods for the measurement of hiding power have been presented, and considerable progress has been made toward establishing a standard method for the measurement of tinting strength. At present, the work of the committee is an effort to establish a satisfactory hiding power method, and the indications are that the results are quite satisfactory.

While the committee and cooperators in these studies realize that these two properties of pigments are not identical, some confusion has arisen due to the synonomous use of the two terms. At various times a relationship between tinting strength and hiding power has

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[1] Research Division, The New Jersey Zinc Co., Palmerton, Pa.

been suggested, and last year R. L. Hallett[1] presented data indicating hiding power to be a straight-line function of tinting strength.

It is the purpose of this paper to present a summary of the hiding power methods used in the laboratory of the New Jersey Zinc Co., pointing out the reproducibility that can be obtained, and showing the correlation between these hiding power results and tinting strength.

## METHODS

In this study, the authors have attempted to abide as closely as possible by the working definition of hiding power as set up by Subcommittee VIII. Any exceptions have been noted in the summary of methods. The definition of hiding power is as follows:

> The hiding power of a paint is measured by that area of impervious black-and-white background which can be covered by a specified quantity of paint and bring the brightness over the black background to within 98 per cent of that over the white background. The white portion of the background shall have a brightness of 80 per cent ± 2 per cent and the black shall be less than 8 per cent brightness. It shall be specified whether the determination is made before or after the paint has dried.

The only tests, conforming to the definition, of which we have knowledge and which have not been included in this correlation are the ones described by Bruce[2] and the black-and-white or practical cryptometer of Pfund.[3] Bruce's method, if used according to the definition of hiding power, should not differ very greatly from the results obtained on the photo-electric cryptometer or the mechanical brush-out method. It was necessary to eliminate the practical cryptometer, since our instrument was prepared some time ago and does not conform to the specifications for the brightness of the black-and-white backgrounds.

For the sake of brevity, no descriptions of methods are given, reference being made to the article containing a complete discussion of the test. The following summary points out wherein the test deviates from the definition of hiding power. A table is included showing the reproducibility that may be obtained with each of the methods.

The absolute cryptometer[4] uses a visual obliteration rather than the defined photometric end point and brings the brightness over the black background to within the visible contrast limit of the ultimate brightness of the paint rather than to 98 per cent of the brightness of the paint over a white background of 80 per cent brightness.

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[1] R. L. Hallett, "Hiding Power and Tinting Strength of White Pigments," *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part II, p. 895 (1930).

[2] H. D. Bruce, "A Photometric Method for Measuring the Hiding Power of Paints," Bureau of Standards *Technologic Paper No. 306*.

[3] A. H. Pfund, "Hiding Power Measurements in Theory and Application," *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part II, p. 878 (1930).

[4] A. H. Pfund, "The Hiding Power of White Pigments and Paints," *Journal*, Franklin Inst., Vol. 188, p. 675, November, 1919.

The Gardner brush-out method[1] calls for a visual grading rather than the defined photometric end point, but conforms in all other respects to the definition.

The modified mechanical brush-out method[2] deviates from the definition only in that a visual end point is used rather than the photometric one.

The mechanical brush-out method[3] using the photometric end point, measures the film thickness having a brightness of 98 per cent of the ultimate brightness of the paint. The photometer used has been carefully recalibrated and the results obtained and reported in this study vary slightly from those reported in the original work.

The photo-electric cryptometer[4] meets the specifications of the definition in all respects and is being reported on by A. H. Pfund at this meeting, so no details are given here.[5]

The figures tabulated below are indicative of the degree of reproducibility obtainable with the various methods. These data are believed to be representative and were selected from determinations made on unknown paints in the course of routine work.

| Operator A, sq. ft. per gal. | | Reproducibility of Test by Single Operator, per cent | Operator B, sq. ft. per gal. | Reproducibility of Test Between Operators, per cent |
|---|---|---|---|---|
| Determination No. 1 | Determination No. 2 | | | |
| ABSOLUTE CRYPTOMETER | | | | |
| 187 | 198 | 5 | 187 | 2.5 |
| 264 | 261 | 1 | 269 | 2.5 |
| GARDNER BRUSH-OUT METHOD | | | | |
| 222 | 213 | 4 | 183 | 15 |
| 189 | 167 | 12 | 175 | 1 |
| MODIFIED MECHANICAL BRUSH-OUT METHOD | | | | |
| 262 | 246 | 6 | ... | ... |
| 166 | 162 | 2.4 | ... | ... |
| MECHANICAL BRUSH-OUT METHOD | | | | |
| 418 | 408 | 2.4 | ... | ... |
| 388 | 369 | 5 | ... | ... |

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[1] H. A. Gardner, G. G. Sward, S. A. Levy, "A Practical Brush-Out Test for Hiding Power of Paints," *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part II, p. 891 (1930).

[2] G. S. Haslam, "Modification of the Spinning Film Hiding Power Method," *Industrial and Engineering Chemistry*, Analytical Edition, Vol. 2, p. 319, July, 1930.

[3] G. S. Haslam, "Evaluation of the Contrast Obliterating and Brightening Power of White Pigments," *Industrial and Engineering Chemistry*, Analytical Edition, Vol. 2, January, 1930, p. 69.

[4] A. H. Pfund, "The Hiding Power of White Pigments and Paints," *Journal*, Franklin Inst., Vol. 188, p. 675, November, 1919.

[5] A. H. Pfund, "The Photo-Electric Cryptometer," see p. 876.

## PAINTS

Paints A to I are single pigment paints made up in a mill gloss vehicle at different pigment-vehicle ratios so that all paints have the same consistency as determined on the Gardner mobilometer. Analysis of the paint is given below:

| PAINT | PIGMENT | PIGMENT, PER CENT | VEHICLE, PER CENT |
|---|---|---|---|
| A | Carbonate White Lead | 67.8 | 32.2 |
| B | Zinc Oxide | 49.0 | 51.0 |
| C | Lithopone | 58.3 | 41.7 |
| D | Barium-Titanium Pigment | 60.6 | 39.4 |
| E | Titanated Lithopone | 57.3 | 42.7 |
| F | Calcium-Titanium Pigment | 53.6 | 46.4 |
| G | High Strength Lithopone | 57.4 | 42.6 |
| H | Zinc Sulfide | 54.6 | 45.4 |
| I | Titanium Oxide | 48.8 | 52.2 |

TABLE I.—RESULTS OF HIDING POWER DETERMINATIONS, SQ. FT. PER LB. OF PIGMENT.

| Paint | Pigment | Methods Using Photometric End Point | | | | Methods Using Visual Grading for Establishing Complete Hiding | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Photo-Electric Cryptometer | | Mechanical Brush-out | | Gardner Brush-out | Modified Mechanical Brush-out | Absolute Cryptometer | Cryptometer 60–40 Pigment-Vehicle Ratio | |
| | | 99 per cent | 98 per cent | 99 per cent | 98 per cent | | | | Original Brush-out | at 90 per cent |
| A | Carbonate White Lead | 14.7 | 19.9 | 14.0 | 18.0 | 13.6 | 13.6 | 15.2 | 14.6 | 11.6 |
| B | Zinc Oxide | 25.5 | 30.2 | 24.0 | 32.0 | 19.9 | 19.9 | 22.0 | 18.6 | 18.6 |
| C | Lithopone | 24.0 | 34.1 | 26.3 | 34.0 | 20.2 | 20.2 | 23.0 | 19.8 | 23.7 |
| D | Barium-Titanium Pigment | 25.0 | 37.3 | 27.0 | 43.0 | 22.0 | 22.0 | 23.3 | 18.0 | 25.6 |
| E | Titanated Lithopone | 27.5 | 43.0 | 35.0 | 43.0 | 23.8 | 23.0 | 30.7 | 26.3 | 28.8 |
| F | Calcium-Titanium Pigment | 30.7 | 43.7 | 35.5 | 56.0 | 25.9 | 25.9 | 28.9 | 24.1 | 31.3 |
| G | High Strength Lithopone | 28.0 | 50.2 | 35.0 | 47.0 | 26.9 | 25.8 | 34.0 | 30.2 | 36.1 |
| H | Zinc Sulfide | 53.0 | 76.1 | 53.0 | 71.0 | 43.2 | 36.8 | 44.8 | 41.2 | 47.7 |
| I | Titanium Oxide | 84.5 | 132.7 | 84.0 | 128.0 | 54.4 | 52.0 | 62.8 | 43.7 | 67.9 |

The cryptometer values of these same pigments at equal pigment-vehicle ratios have also been determined.

## RESULTS

The results obtained are given in Table I. In the first group are placed the results of those tests in which the defined end point is determined by photometric means. The second group includes those tests in which the obliteration of the contrast is graded by the eye. All results are calculated to square feet per pound of pigment for the purpose of comparison. All tests herein noted measure wet hiding power.

## DISCUSSION OF RESULTS

Considering first the results obtained using the photometric end point of the definition, we find a fairly satisfactory agreement between the photo-electric cryptometer and the mechanical brush-out method. However, when these results are compared with the results of the tests using the visual end point, it is obvious that they are not of the same order of magnitude. The conclusion must be drawn, therefore, that the eye is capable of detecting a closer contrast than 98 to 100, the assumption made in formulating the definition.

To establish the correct definition will require considerably more study and investigation, but some preliminary work has been done and is reported here in order to point out the type of results to be obtained on the photo-electric cryptometer. In Table II are recorded the values of hiding power assigned to these nine paints when some contrast ratios other than 98 per cent are used. Plotting the curves and interpolating to 99 per cent, values were obtained which much more nearly approach the order of magnitude given by those tests employing the visual end point. These values appear in Table I.

TABLE II.—RESULTS OF HIDING POWER DETERMINATIONS USING VARIOUS CONTRAST RATIOS.

| Paint | Pigment | Hiding Power with Various Contrast Ratios | | | | |
|---|---|---|---|---|---|---|
| | | 98 per cent | 98.33 per cent | 98.46 per cent | 98.85 per cent | 99.05 per cent |
| A | Carbonate White Lead | 19.9 | 17.5 | 17.0 | 15.85 | 14.5 |
| B | Zinc Oxide | 30.2 | 29.0 | 27.6 | 27.3 | 24.7 |
| C | Lithopone | 34.1 | 33.7 | 33.0 | 28.0 | 22.4 |
| D | Barium-Titanium Pigment | 37.3 | 33.1 | 31.8 | 29.0 | 23.8 |
| E | Titanated Lithopone | 43.0 | 39.5 | 37.7 | 32.0 | 26.0 |
| F | Calcium-Titanium Pigment | 43.7 | 40.3 | 37.0 | 34.2 | 29.5 |
| G | High-Strength Lithopone | 50.2 | 44.0 | 42.8 | 33.8 | 27.0 |
| H | Zinc Sulfide | 76.1 | 75.3 | 74.1 | 62.9 | 48.7 |
| I | Titanium Oxide | 132.7 | 129.2 | 117.3 | 94.2 | 81.5 |

While we do not attempt to establish 99 per cent as the proper end point on the basis of the meager work reported here, we are reporting this value with some confidence that it more nearly approaches the correct hiding power than does the value obtained at the 98-per-cent contrast ratio. These data are reported here to point out the need for cooperative work to establish satisfactorily the contrast that most nearly represents what the average human eye will be just able to detect. The eye is obviously more sensitive than the definition assumes. The application of the extremely

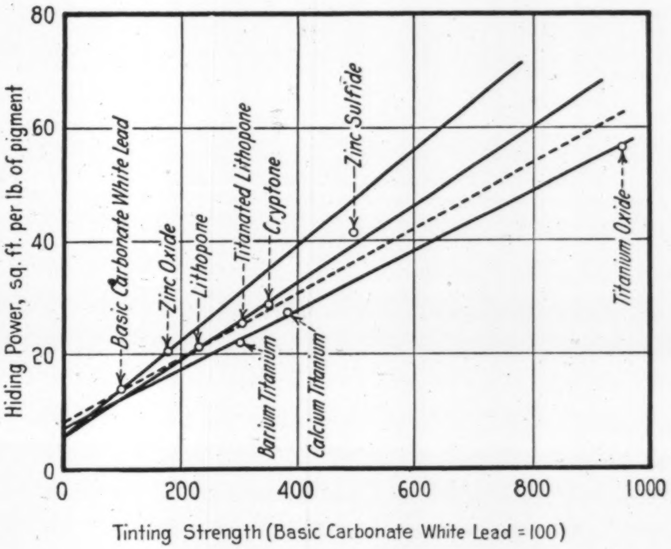

FIG. 1.—Average Visual Hiding Power Tests *versus* Average Tinting Strength Tests.

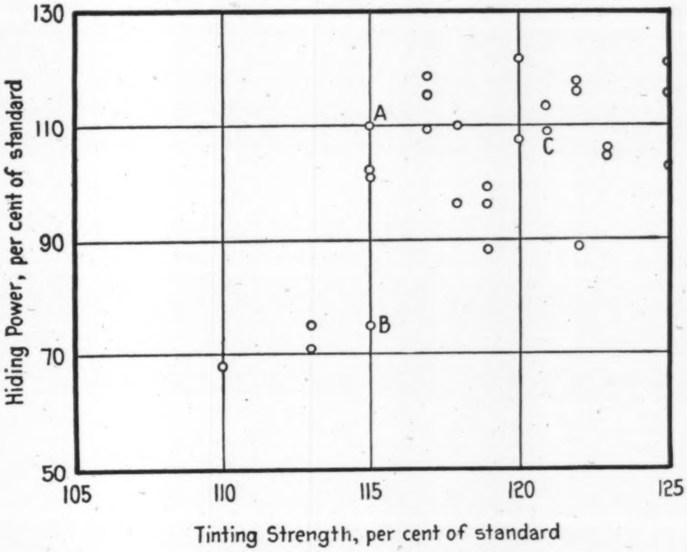

FIG. 2.—Comparison of Hiding Power and Tinting Strength Within Single Type of Pigment.

accurate and reproducible method of the photo-electric cryptometer depends on satisfactorily establishing this contrast ratio.

The pigments from which these nine paints were formulated are the same as were used in the cooperative investigations of the tinting strength method, so we have available for each pigment a tinting strength value which represents the average of twelve determinations, some of which are themselves averages of several determinations. While there is a variation of from 5 to 20 per cent between laboratories, the average can be assumed to be accurate to at least 5 per cent. The reproducibility of the various hiding power methods is shown elsewhere in the paper.

In order to reduce as much as possible the factor of reproducibility, the visual tests have been averaged and the data are given below and are plotted in Fig. 1:

| | TINTING STRENGTH, BASIC CARBONATE WHITE LEAD EQUAL TO 100 | HIDING POWER, SQ. FT. PER LB. OF PIGMENT |
|---|---|---|
| Basic Carbonate White Lead | 100 | 14.1 |
| Zinc Oxide | 176 | 20.6 |
| Lithopone | 233 | 21.1 |
| Barium-Titanium Pigment | 307 | 22.4 |
| Titanated Lithopone | 311 | 25.8 |
| Calcium-Titanium Pigment | 382 | 27.6 |
| High-Strength Lithopone | 357 | 28.9 |
| Zinc Sulfide | 496 | 41.6 |
| Titanium Oxide | 956 | 56.4 |

Study of these data shows that while over the large range of pigments employed, tinting strength and hiding power both increase, this relation is not a straight line within the limit of error of the tests employed as evidenced by the dotted line drawn through the points on Fig. 1. Perhaps the most interesting observation is that there is a tendency for similar pigments to group themselves in a more or less regular manner in this comparison of averages. As a result, it is more nearly possible to draw a straight line through pigments of the same type, that is, all titanium pigments on one curve and all zinc sulfide pigments on another, with zinc oxide and basic carbonate white lead on a third. We have plotted each individual test in a similar manner and have found this tendency for similar pigments to group themselves to exist in every case. In presenting these data we do not subscribe to the idea that even pigments in the same class show a straight-line relationship between tinting strength and hiding power but suggest that this certainly appears to be a much closer approach to the true state of affairs than an assumption of a single straight-line relationship for all white pigments.

Our hesitancy to subscribe to a straight line relationship between these two properties is based not on the curves submitted which contain only a few samples of pigments of the same type but rather on more extensive data gathered in the course of other investigations. Figure 2 shows the hiding power - tinting strength values for a series of experimental pigments, all of which have the same chemical composition, and all have a color and brightness that would meet commercial specifications. In this figure it is obvious that two pigments even of the same chemical composition such as A and B may have equal tinting strengths with as much as 30 per cent difference in hiding power. The reverse may also be true as shown by pigments A and C; that is, two pigments may have equal hiding power but differ quite appreciably in tinting strength. It is data of this type which are in part responsible for our opinion that it is not probable that any definite relationship between hiding power and tinting strength exists even in a group of similar pigments.

We are not in a position to explain conclusively these facts, although it has been pointed out before that the brightness - film thickness curves for different pigments[1,2] are quite different; that titanium pigments have what we termed a high brightening power but not a correspondingly high contrast-obliterating power. It is this factor of brightening power that shows one paint to be superior to another when a one-coat comparison is made, but only equal or possibly inferior to that paint in a two or three-coat comparison. Brightening power is possibly unduly influencing the determinations made by photometering the end point according to the definition. This possibility makes it important that a contrast ratio be chosen that will eliminate the factor of brightening power and permit the determination of contrast-obliterating or hiding power. It is felt that this factor is also responsible for the discrepancies that enter into any effort to establish a straight-line relationship between hiding power and tinting strength, and that when pigments with essentially the same shape brightness - film thickness curves are compared, a much better agreement between the two tests can be found. But since it is possible to vary the factors influencing brightening

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[1] G. S. Haslam, "Evaluation of the Contrast Obliterating and Brightening Power of White Pigments," *Industrial and Engineering Chemistry*, Analytical Edition, Vol. 2, p. 69, January, 1930.

[2] D. L. Gamble and A. H. Pfund, "Experimental Determination of Brightness - Film Thickness Curves of Wet Paints," *Industrial and Engineering Chemistry*, Analytical Edition, Vol. 2, p. 63. January, 1930.

power, it is felt that to consider tinting strength as having a straight-line relationship even in any one class of pigments is hazardous.

As has been pointed out many times in the past, the reduction in the brightness of a given pigment results in a modification of the brightness - film thickness curve of that pigment so as to increase its hiding power. The effect of a reduction in brightness on tinting strength must necessarily result in a decrease in this property. The mere fact that these two properties of a pigment are affected in opposite ways by this modification of the pigment makes the existence of a straight-line relationship between these properties theoretically impossible.

It has been found possible to determine hiding power with the same ease of manipulation as tinting strength and with as good or better reproducibility. Since it is desirable to avoid any necessity of postulation, except for the scientific understanding of the facts, it is suggested that hiding power should be measured and specified where hiding power is meant rather than tinting strength.

# DISCUSSION

Mr. R. L. Hallett[1] (*presented in written form*).—This paper is based on the examination of nine different white pigments selected and distributed by Subcommittee VIII on Methods of Analysis of Paint Materials of the Society's Committee D-1 on Preservative Coatings for Structural Materials. The members of the subcommittee have tested the tinting strength of these nine pigments, using the tentative tinting strength method being submitted by Committee D-1 this year,[2] and the tinting strength results obtained by Subcommittee VIII are reported by Haslam and Gamble on page 866 of the paper.

Haslam and Gamble have made paints with these nine pigments and have measured the hiding power of the paints by a number of different methods using brush-out tests and cryptometer tests in which the readings have been made visually and also photometrically.

Hiding power results are reported by Haslam and Gamble in Table I of the paper. An examination of this table indicates very wide variations in the hiding power results although in any one test the figures for the different pigments seem to bear a somewhat consistent relation with each other which indicates that the variations in the results obtained in the different tests probably are due largely to the variations in the methods used. Therefore perhaps a more accurate conception of hiding power relations can be gained by separately considering the results obtained by the different methods instead of averaging the results of the tests.

In the accompanying Fig. 1 are shown plots and lines indicating the relation between tinting strength and hiding power, using the tinting strength figures reported by Haslam and Gamble and separately plotting the tinting strength figures with the hiding power figures obtained by Haslam and Gamble in the different brush-out tests read both visually and photometrically. These plotted points and the lines connecting them indicate very clearly that the relation between tinting strength and hiding power is a straight line. The failure of some of the points to fall exactly on the straight lines must obviously be due to experimental error rather than to any failure of the straight-line theory. This is particularly true of titanox B, as it is evident that Haslam and Gamble have failed to develop the true hiding power of this pigment. I have illustrated this point by using the figures from the brush-out tests because in our experience the cryptometer tests are much harder to control and are more liable to introduce experimental error.

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[1] Chemist, National Lead Co., Brooklyn, N. Y.

[2] Tentative Method of Test for Tinting Strength of White Pigments, *Proceedings*, Am. Soc. Testing Mats., Vol. 31, Part I, p. 797 (1931).

As tinting strength is a direct measure of hiding power and as tinting strength determinations may easily and quickly be made it would seem desirable to measure the hiding power of the white pigments by determining their tinting strength.

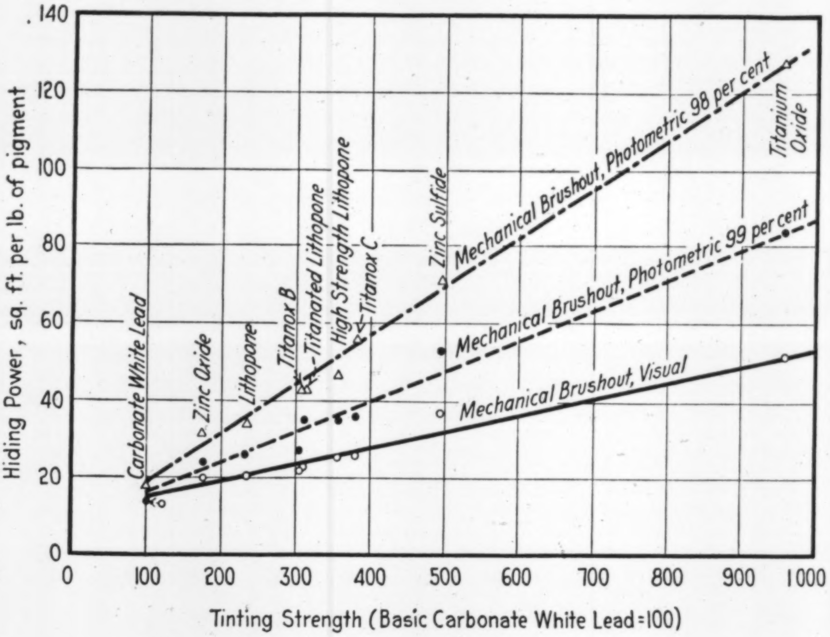

FIG. 1.—Hiding Power Tests *versus* Tinting Strength Tests.

In our paper on "Hiding Power and Tinting Strength of White Pigments" presented before the Society in 1930,[1] we reported hiding power figures for substantially the same white pigments as are included in the tests reported by Haslam and Gamble. The results obtained in these different tests differ greatly but perhaps we can clarify this study and explain the cause of the variation.

The visually read brush-out tests used by Haslam and Gamble were made by applying the paints until the operator considered that the black and white background was completely hidden. In such tests the operator naturally does not stop until he has complete visual hiding and the amount of paint used in excess of that needed to give complete hiding cannot be determined and of course the excess paint lowers the hiding power results. We have made similar tests many times and have found that we unconsciously apply more paint than is required to give complete hiding and of course we never know how much excess paint has been put on. As white paint is applied to a black surface by arithmetical additions, each additional coat increases the total hiding of the previous coats by rapidly decreasing substantially logarithmic increments, and for this reason a slight addition of hiding after practically complete visual hiding has been obtained is accompanied by very great increase in the amount of paint used and therefore very great decrease in the calculated hiding power of the paint. I am inclined to think that this may be the principal reason why the visually read brush-out tests reported by Haslam and Gamble have given abnormally low figures for the hiding power of the white pigments.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part II, p. 895 (1930).

The photometrically read brush-out tests reported by Haslam and Gamble have given much higher hiding power results apparently because when the end point is read with a photometer an excess of paint is not put on and the end point is more accurately determined.

In determining the hiding power results which we reported in 1930 we used a visually read brush-out test but we applied a large number of thin coats until complete visual hiding was obtained and in this way avoided any large excess of paint because the last coat required to give visual hiding could readily be determined by observation of previous coats.

It is interesting to note that the hiding power figures we reported in 1930 are very close to the average of the 98-per-cent and the 99-per-cent photometrically read brush-out results reported by Haslam and Gamble, or in other words, the hiding power figures which we obtained with the visually read brush-out tests in which the end point was accurately determined correspond fairly well to the brush-out results reported by Haslam and Gamble using accurately determined photometrically read end point in which the end point is considered as $98\frac{1}{2}$-per-cent obliteration of the contrast ratio. These fairly close checks would seem to indicate that hiding power figures obtained by photometrically read brush-out tests using $98\frac{1}{2}$-per-cent obliteration of contrast ratio as the end point and also obtained by visually read multiple coat brush-out tests with the end point accurately determined are reasonably correct and may be accepted as representing the practical hiding of these white pigments.

To further demonstrate these hiding power figures, we prepared panels with black and white background over which we applied paints in which single white pigments were used. The paints selected were made with zinc oxide, titanox B, titanium oxide, lithopone and zinc sulfide with pale boiled linseed oil as the vehicle. One coat of each paint was spread on a black and white panel at a rate which would give considerable hiding but much less than complete visual hiding. One series was painted with spreading rates carefully controlled to give each panel identically the same calculated hiding based on the hiding power figures we reported in 1930 or the similar figures obtained by averaging the 98-per-cent and 99-per-cent photometrically read brush-out tests reported by Haslam and Gamble. A second series was painted in a similar way but was based on the much lower hiding power figures obtained by Haslam and Gamble in their visually read brush-out tests.

It was found that panels calculated to have the same hiding power based on our 1930 hiding power figures and the 98½-per-cent average photometrically read brush-out figures reported by Haslam and Gamble did show approximately the same degree of hiding. It was also found that the panels based on the much lower visually read brush-out figures reported by Haslam and Gamble did not show the same hiding, the paints made with the higher hiding power pigments showing much less hiding than the paints with the lower hiding power pigments.

These tests with the paints spread to give much less than complete visual hiding, give a fair and accurate comparison of the relative hiding power of the various white pigments and indicate the accuracy of our 1930 results and the similar 98½-per-cent photometrically read results of Haslam and Gamble. I am familiar with the published work of Gamble and Pfund and others, and I have noted the statements of some writers that hiding power shown by different paints spread to give considerably less than complete visual hiding is not an accurate measure of the hiding power obtained when the same paints are spread to give complete visual hiding, but such statements do not appear to be supported by either theoretical or practical evidence and would seem to be contrary to the scientific facts. This is entirely a question of optics and to get a true conception of these relationships the well-understood principles of pure optics must be applied.

In accordance with the laws of optics, it is obvious that two paints which have a certain hiding relationship when spread to give less than complete visual hiding must have a similar relationship when spread in increasingly thicker coats to approach complete hiding.

Let us not befog the issue by a mass of seemingly conflicting technical data, because the questions are fairly simple and if viewed in a practical manner, but still in accordance with the pure optics involved, the seeming inconsistencies can be explained and hiding power figures for the various white pigments can be obtained which are not only theoretically accurate but are also of the greatest practical value.

I am inclined to think that this discussion indicates very clearly that the average of the 98-per-cent and 99-per-cent photometrically read brush-out hiding power figures reported by Haslam and Gamble and the similar hiding power figures which we reported in 1930 approximately represent the practical hiding power of the white pigments and are the best guide for the paint manufacturer who uses these pigments in the practical manufacture of commercial paints.

MR. G. F. A. STUTZ.[1]—It is evident, from Mr. Hallett's discussion of this paper, that he has given it considerable time and careful thought and I should like to discuss each of the points which he raises in some detail.

Mr. Hallett objects to the fact that an average hiding power was used in plotting the curves given in Fig. 1 of the paper. May I point out that only three methods were used in obtaining the average: namely, the Gardner brush-out, the modified mechanical brush-out and the absolute cryptometer. These methods are all so similar and give results so close to one another that the fairness of averaging such results is obvious. This average was drawn merely to decrease the number of curves necessary. However, it is not necessary to average these results in order to point out the discrepancies existing between hiding power and tinting strength. A separate plot of the results obtained by each method shows the same lack of agreement.

Mr. Hallett has selected three methods of hiding power determination and plotted hiding power against tinting strength; he has then drawn an average curve through the points and has assumed this to be a straight line. He claims that the failure of some of the points to fall on the straight line must, obviously, be due to experimental error. Such curves appear to be in fair agreement with the points plotted, but it must be remembered that they represent a very large span in hiding power and tinting strength so that what appear on the curves to be only slight variations actually amount to a great deal more than the experimental error.

Both of the hiding power methods which Mr. Hallett has plotted against tinting strength and for which he has assumed a straight-line relationship, have an accuracy of approximately 5 per cent or better. The tinting strength figures are also probably accurate to within at least 5 per cent. However, an examination of the curve marked "mechanical brush-out photometric 99 per cent" in Mr. Hallett's plot shows lithopone and titanox B to have practically the same hiding power. Titanox B has 25 per cent greater tinting strength than lithopone. It is obvious to us that such a deviation from Mr. Hallett's straight-line theory cannot be explained so simply as by attributing it to experimental error. Similar discrepancies can be pointed out in the case of other points deviating from the curves drawn. There is no reason to suspect that these two hiding powers of titanox B, as defined, have not been determined to within the accuracy of the test methods as stated above. Reasons for the occasional misconception that titanox B has considerably more hiding power than Haslam and Gamble have assigned to it are given in the paper. We must, therefore, repeat that, while there may be a general relationship between tinting strength and hiding power over the entire range, there is no exact relationship which will permit one to accept tinting strength determinations as a measure of hiding power.

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[1] Research Division, The New Jersey Zinc Co., Palmerton, Pa.

Mr. Hallett claims that, in the Gardner brush-out test and the mechanical brush-out test, paint is applied until the black and white background is completely hidden. This, of course, is relative. Sufficient paint is applied in each case until the hiding has reached a point where the operator can no longer detect the underlying background. However, photometric measurements indicate that even when the operator can no longer detect the difference, a difference of approximately 1 per cent still exists. We, therefore, cannot agree that in such tests an excess of paint has been applied nor can we agree that the hiding power figures given by the visually read brush-out tests are abnormally low. The conclusion drawn by Messrs. Haslam and Gamble accordingly seems quite justified, that in the brush-out test the operator can easily detect the underlying background until the black background has been brought up to a brightness within approximately 99 per cent of the white background. The fact that Mr. Hallett's hiding power figures are abnormally high must be attributed in part to the small area used in making his tests, in part to possible differences in illumination from those used by Messrs. Haslam and Gamble, and in part to the use of a multiple coat application (more than eight coats) which has been shown in our laboratory to give higher hiding power than a two or three-coat application of the same thickness.

Mr. Hallett has resorted to a demonstration of partial hiding power to substantiate his own hiding power figures and to attempt to disprove those of Haslam and Gamble. I would refer him to the papers which have been published during the period of the past three years on the subject of the relationship between the brightness and the film thickness of white paints over a black background. In the paper by Messrs. Gamble and Pfund entitled "Experimental Determination of Brightness Film Thickness Curves of Wet Paints,"[1] the authors have concluded that it is impossible to employ a theoretical or empirical formula to express the complicated optical system presented by a suspension of pigment in oil. Messrs. Pfund and Gamble have proved, in this paper, that no obvious conclusion can be drawn or no "well understood principle of pure optics" can be applied in discussing hiding power relationships. If it were true that simple optical laws could be applied to such suspensions, many of the problems involved would have been settled some time before this. It is the fact that such empirical laws cannot be applied that makes it seem to us most desirable to go back to a simple, practical test such as the Gardner brush-out board or the Pfund cryptometer. The fact that there is no definite and simple brightness-film thickness relationship for all pigments makes it obvious that Mr. Hallett's comparison made by painting out panels and comparing pigments at partial hiding power cannot be expected to agree with hiding power figures obtained by Messrs. Haslam and Gamble at complete hiding power. It is obviously a mere coincidence that the hiding power figures in his own paper more nearly agree with these partial hiding brush-outs than do the figures of Haslam and Gamble.

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[1] *Industrial and Engineering Chemistry*, Vol. 2, p. 63 (1930).

# THE PHOTO-ELECTRIC CRYPTOMETER

By A. H. Pfund[1]

## Synopsis

A precision cryptometer involving the use of a photo-electric cell in place of the human eye has been developed and put into use. The instrument yields results which are in conformity with the accepted definition of hiding power. The accuracy of setting is considerably less than 1 per cent.

A year ago a report was presented on some preliminary work dealing with a cryptometer used in conjunction with a photo-electric cell which replaced the human eye.[2] Since then the instrument has been completed and put into actual use. It seems opportune, therefore, to describe the finished instrument.

The essential feature of this newest cryptometer is that its construction and operation are in strictest conformity with the following definition for hiding power as formulated by Subcommittee VIII on Methods of Analysis of Paint Materials of the Society's Committee D-1[3] on Preservative Coatings for Structural Materials:

> The hiding power of a paint is measured by that quantity of a paint that must be applied to a given area of an impervious black and white background, the white portion of which has a brightness of 80 per cent plus or minus 2 per cetn and the black to be less than 8 per cent brightness, so as to bring the brightness over the black background to within 98 per cent of that over the white background and shall be specified as wet or dry.

While it is not improbable that this definition may be modified and augmented, the fact remains that the instrument about to be described is able to measure hiding power in conformity with whatever definition is finally adopted.

The general arrangement (Fig. 1) is essentially the same as that previously presented—the only changes introduced being in the nature of simplifications. The base plate is of glass and consists of black and white portions *B* and *W* whose line of separation runs longitudinally. The upper surfaces are optically flat to within a few wavelengths and co-planar. The usual grooves, *AA*, *OO* and *QQ*, are provided for the purpose of preventing paint from crawling under the points of contact which define the wedge-angle of the upper plate. The latter is of clear glass, optically flat and of the same size as the lower plate. Because of the greater length of these plates, the wedge-angle has been adjusted to an angle whose tangent is 0.004—the wedge angle being defined by a short steel cylinder about 0.5 mm. in diameter, permanently glued to the lower plate at *M*. So as to prevent changes in the wedge thickness due to capillary creeping, pressure is brought to bear on the top plate by means of three screws *Y* (Fig. 2) whose conical lower tips fit into slight depressions introduced immediately

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[1] Professor of Physics, Johns Hopkins University, Baltimore, Md.

[2] A. H. Pfund, "Hiding Power Measurements in Theory and Application," *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part II, p. 878 (1930).

[3] Indianapolis meeting of the subcommittee, April 1, 1931.

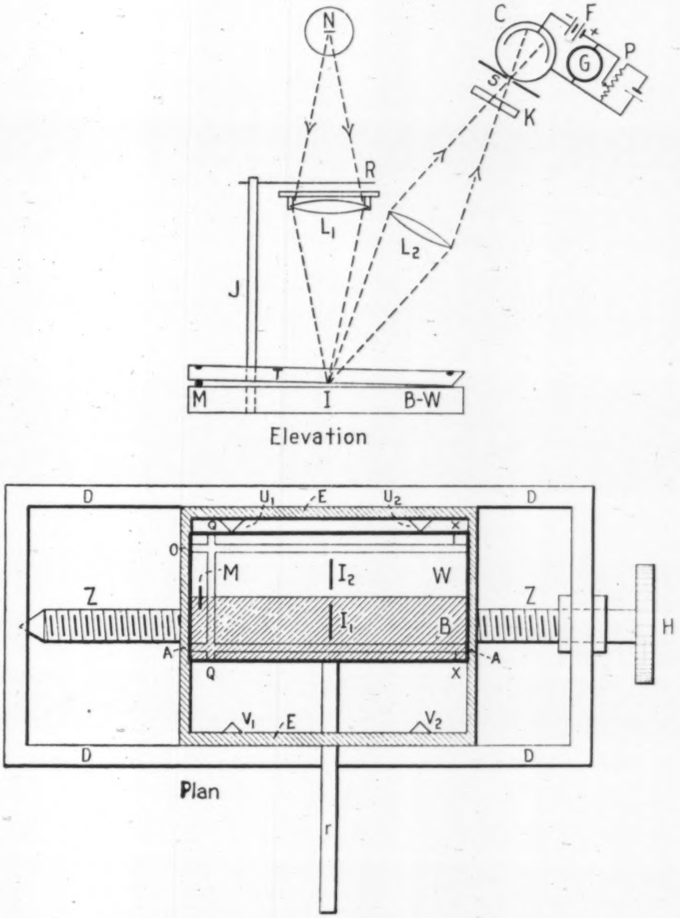

FIG. 1.—Diagrammatic Sketch of Cryptometer.

above the three points of contact $x$, $x$, and $M$. A table, bearing the glass plates may be advanced in the direction of changing thickness of the paint film by means of the screw $Z$ and divided head $H$. The screw has a pitch of about 1 mm. An image, $I$, of the ribbon-filament lamp $N$ is projected normally on the paint film by means of the lens $L_1$. This image, acting as a new source of light, is subsequently focused, by means of a lens $L_2$, on a narrow slit $S$ mounted in front of the photoelectric cell $C$. Ample sensitivity is obtained by connecting this cell (known as the General Electric Co. Type PJ–23) with a 90-volt B-battery $F$, a high-resistance galvanometer $G$ of the Leeds & Northrup H–S type and a simple potentiometric device $P$ for bringing the galvanometer light back on the scale. The full period of the galvanometer is, preferably, of the order of 10 seconds. The filter $K$ is of such a character that the product of the energy distribution of the source times the transmission curve for the filter, times the sensitivity curve of the photo-electric cell yields a curve of the same form as that obtained from the product of the sensitivity of the average normal

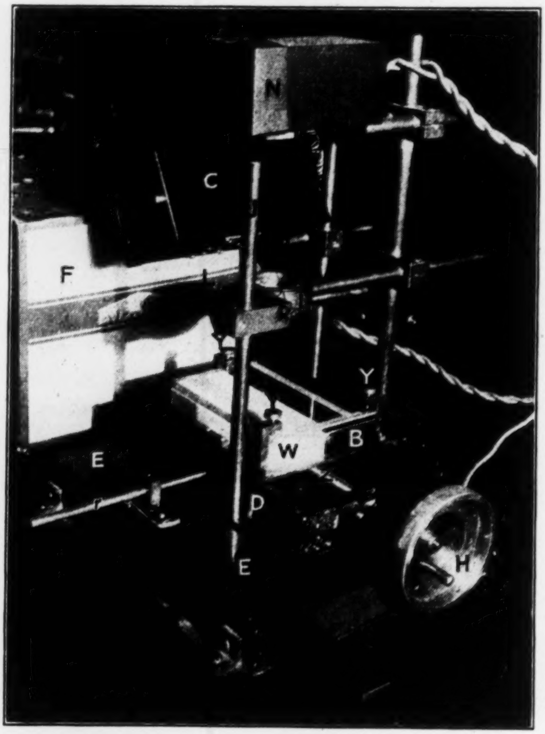

FIG. 2.—Photographic View of Cryptometer.

human eye times the energy distribution for a black-body at a temperature of 6500° K. (average daylight). A second platform and ways have been introduced at *EE* so that a transverse motion between the stops $U_1U_2$ and $V_1V_2$ can be effected. This motion is brought about by means of the rod *r*.

To operate the instrument it is merely necessary to shift the cryptometer so that the filament image $I_1$ is projected on the paint film overlying the black background; next, the galvanometer spot of light is brought back on the scale by means of the potentiometer *P*, and, finally, the cryptometer is shifted quickly by means of the rod *r* so that the filament image $I_2$ is projected on the paint film overlying the white field. Now, a vertical post *J* bears a wire *R* of such dimensions that 2 per cent of the light falling on the aperture at $L_1$ is occulted. Since this post is permanently attached to the cryptometer plates, the wire moves into position automatically when the white half of the base-plate is illuminated. This simply means that the light coming from the white field is reduced by 2 per cent which is the excess brightness of the white field over the black at "complete" hiding. The position of complete hiding will have been located when no change in the galvanometer deflection results upon shifting the cryptometer transversely.

A photographic reproduction of the finished instrument is presented in Fig. 2. While the disposition of parts is not identical with that shown in Fig. 1, the essential parts have been designated by the same letters. It might be added that the screw and nut arrangement by means of which the cryptometer plates are advanced has been so designed that there is no backlash upon reversal of motion.

The actual performance of this instrument exceeds expectations. Below are given cryptometer settings obtained with the galvanometer scale at a distance of but 75 cm. The deflection brought about by the occulting wire which reduced the intensity by 2 per cent was 200 mm. This means that a deflection of 1 mm. corresponds to an intensity change of $\frac{1}{200}$ of 2 per cent. This degree of refinement cannot be approached by the human eye:

| | Readings | |
|---|---|---|
| | Original | Paint Removed and Replaced by Fresh Film |
| Paint A | 26.55 | 26.4 |
| | 26.65 | 26.6 |
| | 26.56 | 26.53 |
| Paint B | 13.62 | 13.65 |
| | 13.62 | 13.59 |
| | 13.60 | 13.55 |

The average deviation from the mean for all of these data taken together is better than one part in 500. Most gratifying is the closeness of the check reading obtained upon replacing the original paint film by a fresh one. It is evident that, upon replacing the cryptometer by a black-and-white background covered with dry paint films, hiding powers of dry paints may likewise be determined.

*Acknowledgment.*—In conclusion the author wishes to express his appreciation of the labors of Mr. D. L. Gamble, who designed the mechanical parts of the instrument shown and who carried out the measurements previously recorded.

# DISCUSSION

MR. G. W. THOMPSON[1] (*presented in written form*).—This is an interesting addition to methods of determining hiding power. The use of the photo-electric cell is novel to the extent that it is applied to the cryptometer but is not novel in its use for the determinations of small differences of brightness or brightness range ratios.

Determinations of hiding power, whether by the cryptometer or by any other method, until a scheme has been worked out scientifically whereby differences in brightness or brightness range ratios of various thicknesses of coating can be used as the basis of calculation to complete hiding or, more scientifically, to the rate in change of hiding or brightness range, are unsatisfactory. If it is denied that each unit of thickness of paint produces a certain definite effect upon hiding, then all attempts to develop instruments for determining hiding power are futile and it becomes a hit and miss proposition.

I know of no one better qualified than Mr. Pfund to make a study of hiding power and to develop a proper equation. I attempted to do this in the discussions at a meeting of our Society last year.[2] If my method of presentation was not sound it seems to me that Mr. Pfund, or someone equally capable, could develop a better exposition.

MR. R. L. HALLETT.[3]—Mr. Pfund deserves great credit for his pioneer work in this field of optics. His adaptation of the photoelectric cell for determining the brightness of paints is of great scientific and practical interest.

Visual comparisons and visual photometric readings have not been altogether satisfactory and have sometimes given wide variations in results obtained by different operators, but the use of the photoelectric cell as suggested by Mr. Pfund will overcome many of these difficulties and enable us to obtain results with scientific accuracy.

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[1] Chief Chemist, National Lead Co., Brooklyn, N. Y.

[2] G. W. Thompson, Discussion on Hiding Power of Paints, *Proceedings*, Am. Soc. Testing Mats., Vol. 30, Part II, p. 906 (1930).

[3] Chemist, National Lead Co., Brooklyn, N. Y.

# METHODS FOR DETERMINING THE PHYSICAL PROPERTIES OF CERTAIN RUBBER COMPOUNDS AT LOW STRESSES

By R. L. Templin[1] and R. G. Sturm[2]

## Synopsis

This paper describes the apparatus and technique developed for determining the physical properties of rubber compounds having moduli of elasticity ranging from 500 to 50,000 lb. per sq. in. In order to obtain the sensitivity and accuracy required for determining the elastic properties of such materials, special loading apparatus and measuring devices had to be developed practically without the assistance of precedent.

The particular physical properties determined were: moduli of elasticity in tension and compression in three principal directions, Poisson's ratio, permanent set, elastic hysteresis, rate of creep or time yield, specific gravity, coefficient of thermal expansion and effects of water absorption. The first two properties were determined for a large number of specimens of rubber compounds of specific gravity **2.4** ± 0.2 while a complete set of determinations was made for only one final compound which is being used in model studies on statically indeterminate structures, particularly arch dams.

## Introduction

Most of the published information relative to the physical properties of rubber compounds has dealt entirely with conditions involving high stresses. Various forms and sizes of test specimens have been used together with different kinds and types of testing apparatus. Furthermore, in view of the absence of the details of the testing procedure used, comparisons of results are at best very difficult.

The selection of a suitable rubber compound for use as a material from which models of statically indeterminate concrete structures (particularly arch dams) could be made depended among other things upon a definite knowledge of the physical properties of the material. For such purposes the range of stresses from 15 lb. per sq. in. in tension to 20 lb. per sq. in. in compression had to be investigated. In addition to the stress-strain relationships, tests were made to define Poisson's ratio, permanent set, elastic hysteresis, rate of creep or time yield, specific gravity, coefficient of thermal expansion, and effect of water absorption.

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## TEST SPECIMENS

Because of the absence of suitable standards for specimens and testing procedure, it was necessary to design both specimens and testing apparatus, as well as to develop a procedure for testing, in order to obtain data on the problem presented.

Two forms of compression specimens were used as shown by (*a*) and (*b*) in Fig. 1. Form (*a*) was 2 in. in diameter and $4\frac{1}{2}$ in. long, while the parallel central portion of (*b*) was 2 in. in diameter and 4 in. long with an overall length of about $6\frac{1}{4}$ in. The form of tension specimen used, Fig. 1 (*c*), had a parallel reduced section $\frac{5}{8}$ in. in diameter and $2\frac{3}{4}$ n. long with enlarged ends $1\frac{1}{4}$ in. in diameter by 1 in. long. Fillets of $\frac{1}{8}$-in. radius were used at the ends of the reduced section of the tension specimens.

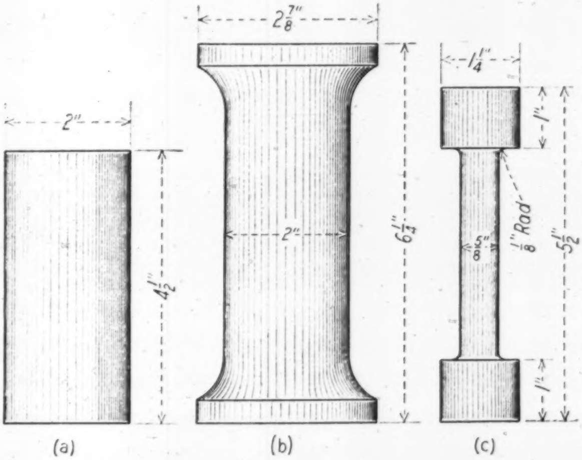

FIG. 1.—Compression and Tension Specimens Used for Rubber Tests.

All specimens were rough sawed using a band saw and then ground to size on a precision grinder using a special grinding wheel (No. 3730 T1—Bakelite Norton).

It was found that the desired dimensions could be obtained within a tolerance of ±0.002 in. Coolants were tried during the grinding operation but best results were obtained without them. Some care, however, was required to see that no overheating occurred in the early stages of grinding when comparatively heavy cuts were made.

When the material to be tested was of such dimensions that the specimens indicated could not be cut directly from it, specimen blanks were built up by cementing together suitable pieces of the sample. This procedure also permitted a study of the directional properties of the samples, using full-size specimens. In so far as could be determined, the presence of the rubber cement used in the built-up specimens had little if any effect on the results obtained.

The form of compression specimen shown in Fig. 1 (*b*) was developed with the idea of avoiding the effects of the restraining action of the bearing plates in the reduced section. That is, it was thought such a specimen would deform laterally more uniformly throughout its reduced section, under compressive loads, than a straight cylindrical specimen. Subsequent tests proved this to be a fact. In general, the form of specimen shown in Fig. 1 (*a*) was used for preliminary exploration of a sample of material and the form shown in Fig. 1 (*b*) for the final or more precise tests.

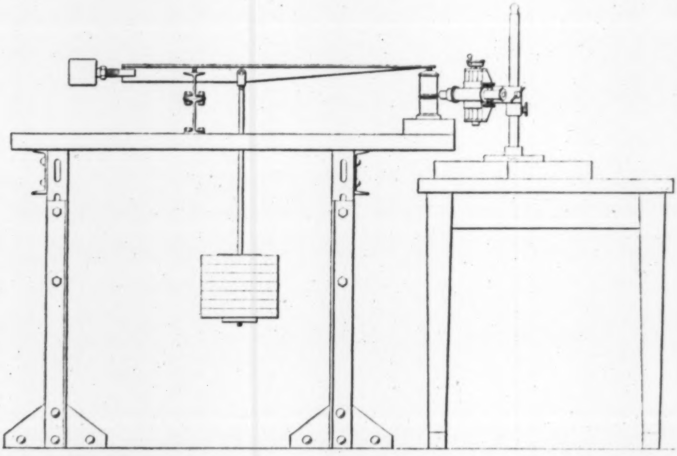

FIG. 2.—Compression Testing Apparatus Used for Rubber Tests.

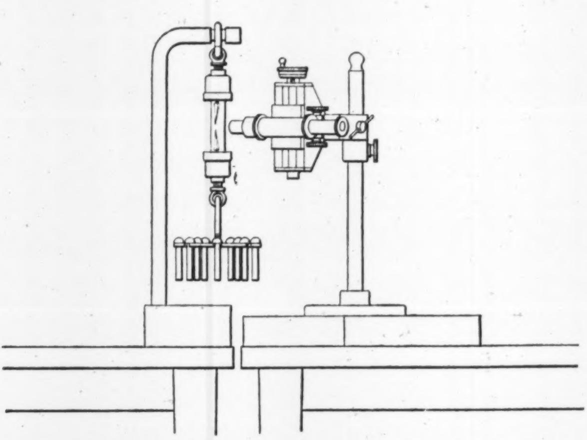

FIG. 3.—Tension Testing Apparatus Used for Rubber Tests.

Specimens were cut from the built-up blanks in three directions. The "A" specimens were cut from the samples so that their longitudinal axes were parallel to the direction of milling and the plane of the sheet or slabs; the "B" specimens were cut so that their axes were normal to the direction of milling but parallel to the plane of the slabs; and the "C" specimens had their axes normal to the plane of the slab samples.

For the flexure-creep tests and thermal expansion tests, rectangular or square specimens 1 by 1 in. or 1 by $1\frac{1}{2}$ in. and 11 in. long were used. The plain cylindrical compression test specimens served for specific gravity determinations, and the thermal expansion specimens for the water absorption tests.

## APPARATUS

In order to apply loads of sufficient accuracy to determine the stress-strain curve in the vicinity of the zero stress region (that is, within 1 lb. per sq. in.) it was necessary to design apparatus for applying stable axial loads to the specimens where the magnitude of applied load could be measured to a small part of a pound.

The compression specimens were loaded by placing standard weights on a pan suspended from the stem of a T-beam as shown in Fig. 2. All bearing points and fulcra were either hardened steel point and cup bearings or hardened steel knife-edge bearings. Lateral movement of the point through which the specimen was loaded was eliminated by carefully leveling the apparatus in two directions and plumbing the specimen before applying load. This device was calibrated before the tests were made, and found to give the load on the specimen within ±0.01 lb. The whole loading device was mounted on a steel channel which in turn rested on a suitable support, thereby making it an entirely independent unit. This arrangement made possible a very stable set-up free from vibration.

The tension specimens were loaded by applying known dead weights directly to the specimen as shown in Fig. 3. Aluminum bell-shaped heads fitting over the ends of the specimen were held in place by means of split nuts which fitted over the shank of the specimen. Centered in each of the heads was a hardened steel eye countersunk, from both sides to provide a knife-edge bearing. The specimen was supported by a hook through the eye of the upper head, while a rack for holding weights was suspended from the eye of the lower head.

For measuring the longitudinal strains, two different devices were used. One consisted of a pair of aluminum wire spurs attached to each side of the specimen with the line of the spurs parallel to the axis of the specimen. Each of these spurs was flattened at one end with a short sharp pin set into the flattened portion. The outer end was tapered and polished to a smooth point. A spur was attached to the specimen by pressing its pin into the specimen at the end of the gage length used. The strains were measured as the change in distance between the adjacent points of the spurs by means of a precision filar microscope.

The other device consisted of an aluminum ring into which the spurs were rigidly set at points diametrically opposite. These rings were placed over the specimen and clamped thereto by means of three set screws spaced about 120 deg. apart. Spacing of the rings and aligning the spurs was readily obtained by means of spacing sleeves and at the same time the proper gage length was obtained without the necessity of measurement. It was thought that this device would give a better average value of the strain than the two sets of spurs above mentioned. The strains again were measured as the change in distance between the points of the spurs.

The microscope used for measuring the change in distance between the points of the spurs was mounted as shown in Figs. 2 and 3. The magnification used was 42X and the total traverse of the instrument was 50 mm. The head of the lead screw was graduated into 100 divisions so that parts of a division could be readily estimated to the nearest 0.2 division which corresponded to 0.002 mm. or 0.00008 in. Calibration of the instrument throughout its range showed an accuracy within the limits just indicated.

The lateral strains in both the compression and tension specimens were measured by the use of aluminum foil bands. These bands were 0.002 in. thick, about $\frac{1}{4}$ in. wide and long enough to go around the specimen and overlap about $\frac{1}{4}$ in. Each of these was held in place with a small rubber band. Free sliding of the aluminum bands was insured by lubricating them with zinc stearate.[1] Fine scratches were put on the bands near their ends and the change in distance between them was measured with one of the filar microscopes properly supported to traverse in a horizontal plane, as shown in Fig. 2 for a compression specimen.

Checks on the modulus and creep tests made on the compression and tension test specimens were made using simple beam specimens loaded at the center. In these tests, changes in deflection were measured with a test dial indicator reading to 0.001 in.

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[1] A few tests were made to determine the effect of using zinc stearate as a lubricating medium between the aluminum bands and the rubber specimens. These tests were not very extensive but the zinc stearate was found to decrease the coefficient of friction from about 0.7 to about 0.2. This appeared to have just enough resistance to prevent the bands sliding down the specimens when they were tested in the vertical position, and at the same time decreased the frictional resistance by about 71 per cent.

In making the thermal expansion measurements, two microscopes rigidly mounted approximately 11 in. apart on a suitable support were used. The change in distance between two reference marks 11 in. apart on the specimens was determined by comparison of the distance between these marks with the distance between two other reference marks on an Invar steel bar. Observations were repeated until three readings agreed within ±0.005 mm. (±0.0002 in.). The value of the difference of the averages of two sets of readings assuming the Invar bar remained at constant length, would give the change in length of the specimen. With the above tolerances two sets of readings would always agree within 0.010 mm. (±0.0004 in.) which is equivalent to ±0.00004 in. per inch.

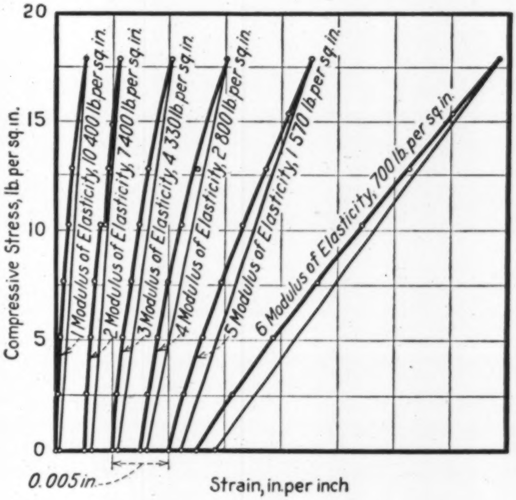

FIG. 4.—Compressive Stress-Strain Curves of Various Rubber Compounds, Specimens 2 by 4½ in.

## DISCUSSION

Two types of tests were made. A reconnaissance determination of the modulus of elasticity, Poisson's ratio, and permanent set was made for a number of different compounds and a complete set of measurements was made on one compound.

In all tests involving stress-strain measurements, the temperature was maintained as nearly constant as possible and did not vary more than ±2° C. for any test and during most tests varied less than

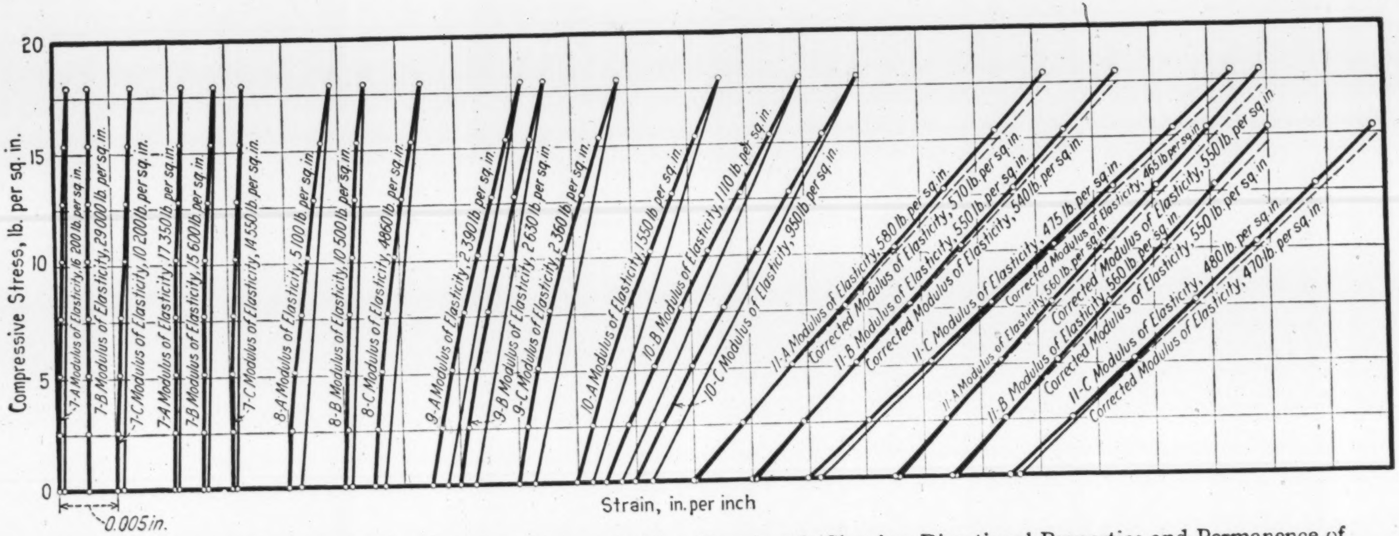

FIG. 5.—Compressive Stress-Strain Curves of Various Rubber Compounds Showing Directional Properties and Permanence of Laminated Specimens.

±1° C. In the first group of tests, load was applied in regular increments until the maximum was reached at which time the entire load was removed and the zero reading taken.

The data from the reconnaissance tests are shown in Figs. 4 and 5. In Fig. 4 the stress-strain curves for specimens cut from cylindrical samples are shown. The value of the modulus of elasticity indicated for each curve is the slope of the secant drawn through the intersection of the stress-strain curve with the zero stress line and the point on the curve at 10 lb. per sq. in. The values for lateral strain are not shown with these curves. The values of Poisson's ratio and permanent set, together with the values of modulus of elasticity for the samples indicated are shown in Table I.

TABLE I.—PROPERTIES OF DIFFERENT SAMPLES.

| Specimen | Corrected Modulus of Elasticity lb. per sq. in. | Poisson's Ratio | Permanent Set, per cent of Maximum Strain | Specimen | Corrected Modulus of Elasticity, lb. per sq. in. | Poisson's Ratio | Permanent Set, per cent of Maximum Strain |
|---|---|---|---|---|---|---|---|
| No. 1 .......... | 10 400 | 0.43 | 13.1 | No. 10a ........ | 1 550 | 0.48 | 8.3 |
| No. 2 .......... | 7 400 | 0.26 | 23.4 | No. 10b ........ | 1 110 | 0.51 | 8.3 |
| No. 3 .......... | 4 330 | 0.48 | 9.5 | No. 10c ........ | 950 | 0.47 | 8.3 |
| No. 4 .......... | 2 800 | 0.43 | 6.9 | | | | |
| No. 5 .......... | 1 570 | 0.51 | 8.4 | No. 11a ........ | 570 | 0.50 | 0.5 |
| No. 6 .......... | 700 | 0.49 | 6.4 | No. 11b ........ | 540 | 0.46 | 0.5 |
| | | | | No. 11c ........ | 465 | 0.48 | 0.5 |
| No. 7a .......... | 16 200 | 0.48 | 22.0 | | | | |
| No. 7b .......... | 29 000 | 0.27 | 22.0 | No. 11a[a] ....... | 550 | .... | 1.2 |
| No. 7c .......... | 10 200 | 0.17 | 22.0 | No. 11b[a] ....... | 550 | .... | 1.2 |
| | | | | No. 11c[a] ....... | 470 | .... | 1.2 |
| No. 7a[a] ......... | 17 300 | 0.42 | 20.9 | | | | |
| No. 7b[a] ......... | 15 600 | 0.36 | 20.9 | No. 11a ........ | 620 | .... | 1.2 |
| No. 7c[a] ......... | 14 550 | 0.26 | 20.9 | No. 11b ........ | 530 | .... | 0.4 |
| | | | | No. 11c ........ | 470 | .... | 0.4 |
| No. 8a .......... | 5 100 | 0.40 | 23.1 | | | | |
| No. 8b .......... | 10 500 | 0.11 | 23.1 | No. 11a[b] ....... | 610 | .... | 1.3 |
| No. 8c .......... | 4 860 | 0.20 | 23.1 | No. 11b[b] ....... | 515 | .... | 5.4 |
| | | | | No. 11c[b] ....... | 445 | .... | ... |
| No. 9a .......... | 2 390 | 0.45 | 16.2 | | | | |
| No. 9b .......... | 2 630 | 0.47 | 16.2 | No. 11a[c] ....... | 630 | .... | ... |
| No. 9c .......... | 2 360 | 0.37 | 16.2 | No. 11b[c] ....... | 600 | .... | ... |

[a] Specimens subjected to hydrostatic pressure of 5000 lb. per sq. in. for 8 to 10 hours.
[b] Results of tension tests.
[c] Results of flexure tests.

Figure 5 shows the stress-strain curves for the specimens cut from blocks made by cementing slabs together. The value of the modulus of elasticity was determined as before. The curves marked "A," "B" and "C" correspond to the specimens cut from the block in the "*A*," "*B*" and "*C*" directions, respectively. Specimens No. 7 and No. 11 are shown twice. In both cases the group of curves to the right represents the data obtained from a retest of the specimen after it had been subjected to a hydrostatic pressure of 5000 lb. per sq. in. for a period of 8 to 10 hours. The degree to which the specimen maintained its original modulus is an indication of the permanence of the material. For specimen No. 11, the modulus was so low that the lateral bulge due to Poisson's ratio was sufficient to change the area of cross-section to such an extent that it was advisable to make corrections for this increase in diameter. The magnitude of the final diameter at any load may be determined from:

$$D' = D\left(1 - \mu \frac{S}{E}\right)$$

where $D$ = the original diameter,
$D'$ = the final diameter,
$S$ = the longitudinal stress (+ for tension and − for compression),
$\mu$ = Poisson's ratio, and
$E$ = modulus of elasticity.

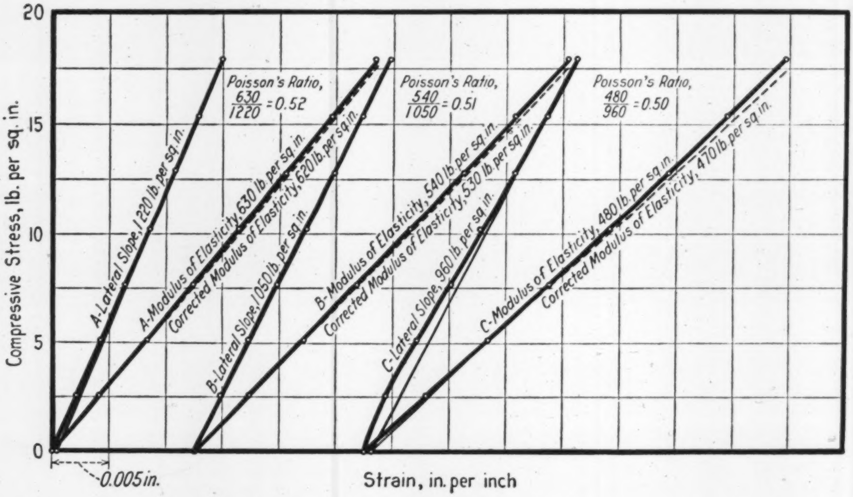

FIG. 6.—Compressive Stress-Strain Curves for Rubber Compound No. 11, Specimens 2 by $4\frac{1}{2}$ in.

Corrections in the stresses ($P/A$) were made on this basis and the resulting values shown by dotted lines in Fig. 5 for specimens of the No. 11 group. For specimens whose modulus was greater than 1000 lb. per sq. in., this correction was practically negligible.

Figure 6 shows the compressive stress-strain curves for one of the samples. It should be mentioned here that this material is exactly the same as specimen No. 11 shown in Fig. 5. The values of lateral strain and longitudinal strain have both been plotted to the same scale. The value of Poisson's ratio then would be the ratio of the slopes of the two curves. In this way, there is a tendency to eliminate erratic readings. Stress-strain curves for tension tests are very similar to these but the lateral strain could not be determined as accurately because of the small diameter.

The values of lateral strain shown in Fig. 6 were all based on measurement made at a section midway between the ends of the specimens. The restraining effect of the bearing cap at the top of the specimen and of the ring connection for holding the spurs was found to have an appreciable effect upon the lateral bulge in the vicinity of the ends. The curves in Fig. 7 show the lateral strains at different places for the same specimens from which the curves in Fig. 6 were obtained. The value of Poisson's ratio obtained from the lateral strain at $1\frac{3}{4}$ in. from the center, 1 in. from the center, $\frac{1}{2}$ in. from the center, and at the center of the specimen, are shown in Table II. Since the longitudinal strain shown in Fig. 6 is an average of the strain occurring throughout the portion of the specimen defined by the centrally located gage length of 2 in.; the average value for lateral slope should in reality be used in determining a value of Poission's ratio of the material.

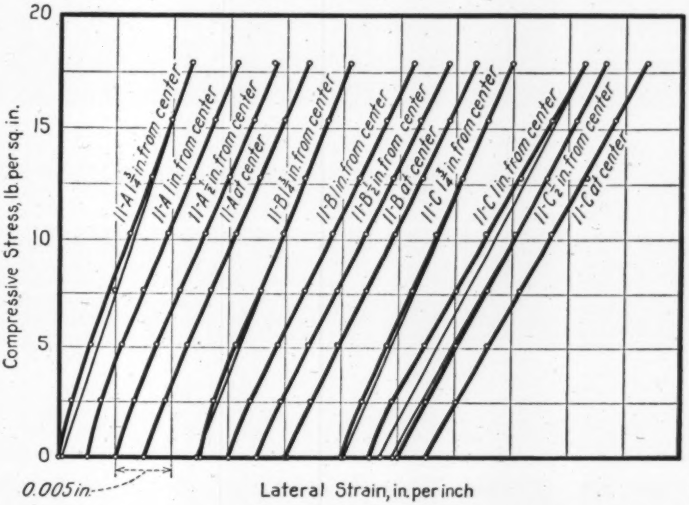

FIG. 7.—Values of Lateral Strain at Various Points Along a 2 by $4\frac{1}{2}$-in. Rubber Specimen.

In the creep tests, the first increment of load was applied to the specimen and maintained constant for a period of time. Increments of load were maintained until no further change in gage length could be detected. For the lower loads no change was noticeable for the first two readings; hence, later readings were not made, but

### TABLE II.—VARIATION IN LATERAL BULGE ALONG SPECIMEN.

| Specimen | Distance of Band from Center of Specimen, in. | Slope of Stress-Strain Curve, lb. per sq. in. | Poisson's Ratio Based on Slopes |
|---|---|---|---|
| FOR STRAIGHT CYLINDRICAL SPECIMEN WITH RING ATTACHMENT | | | |
| A | 1.75 | 1 630 | 0.39 |
| | 1.00 | 1 440 | 0.44 |
| | 0.50 | 1 300 | 0.48 |
| | At Center | 1 250 | 0.50 |
| Average within gage length | ... | ... | 0.47 |
| B | 1.75 | 1 410 | 0.38 |
| | 1.00 | 1 130 | 0.48 |
| | 0.50 | 1 070 | 0.50 |
| | At Center | 1 060 | 0.51 |
| Average within gage length | ... | ... | 0.50 |
| C | 1.75 | 1 240 | 0.39 |
| | 1.00 | 1 000 | 0.48 |
| | 0.50 | 1 000 | 0.48 |
| | At Center | 930 | 0.52 |
| Average within gage length | ... | ... | 0.49 |
| FOR FLARED-END SPECIMEN WITH SPURS | | | |
| C | At Fillet | 1 250 | 0.42 |
| | 1.50 | 1 060 | 0.49 |
| | 1.00 | 1 030 | 0.50 |
| | At Center | 1 015 | 0.51 |
| Average within gage length | ... | ... | 0.50 |

### TABLE III.—VALUES OF CREEP.

| Specimen | Stress, lb. per sq. in. | Increase in Strain, per cent of Initial Strain | Permanent Set, per cent of Maximum Strain |
|---|---|---|---|
| COMPRESSION | | | |
| A | 17.92 | 2.9 | 3.1 |
| B | 17.92 | 3.6 | 3.7 |
| C | 17.92 | 2.4 | 1.6 |
| TENSION | | | |
| A | 15.46 | 2.8 | 4.1 |
| B | 15.46 | 3.0 | 6.0 |
| C | 10.71 | 2.6 | ... |
| FLEXURE | | | |
| B | 13.5 | 9.6 | ... |

for the higher loads readings were taken over periods of time varying from 50 to 100 hours. It was found that the amount of creep for stresses below 10 lb. per sq. in. in both tension and compression was within the limits of experimental error. The amount of creep for the maximum stresses is shown in Table III.

In order to obtain a measure of the errors introduced in strain measurements by the phenomenon of internal friction or hysteresis, the specimens were tested under both increasing and decreasing loads. The stress-strain curves obtained show that the two lines, one for increasing load and the other for decreasing load, ordinarily would fall so close together that no accurate measure of the discrepancy could be obtained. In order to magnify the differences between the strain corresponding to a given value of increasing load and the strain corresponding to the same value of a decreasing load, the method of plotting suggested by L. B. Tuckerman[1] for picking proportional limits has been employed. This consists of plotting the difference between the actual measured strain and the theoretical strain (resulting from some assumed modulus as abscissa), and values of stress as ordinate. In Fig. 8 the values of the assumed modulus are indicated on the vertical line; the arrows on the curves indicate whether the load was increasing or decreasing. It may be noted

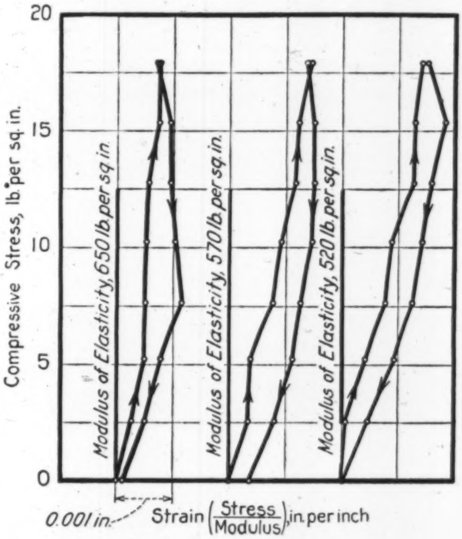

FIG. 8.—Hysteresis Curves of Rubber Compound No. 11.

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[1] Discussion by L. B. Tuckerman of paper by R. L. Templin on "The Determination and Significance of the Proportional Limit in the Testing of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 29, Part II, p. 538 (1929).

from the curves that the difference between the decreasing strain and the increasing strain for the same load was very nearly constant. The values of this difference expressed as a percentage of the maximum strain are as follows:

| SPECIMEN | WIDTH OF HYSTERESIS LOOP, PER CENT OF MAXIMUM STRAIN |
|---|---|
| No. 11a | 1.4 |
| No. 11b | 1.5 |
| No. 11c | 1.4 |

Tests other than those represented in Fig. 8 show that the hysteresis is approximately the same percentage of the maximum strain when the maximum strain to which the material is subjected is somewhat less than that for this particular test.

Specific gravity was determined by weighing carefully measured specimens, or by weighing the same specimens in air and in distilled water. These determinations show that the specific gravity of the different materials all fell within the limits of 2.40 ±0.02.

The determinations of the thermal coefficient of expansion were made by measuring the change in length of specimen between 0 and 20° C. and repeating the measurements for temperatures between 0.6 and 24° C. The value of thermal coefficient of expansion thus determined for the "A" direction is 0.00011 in. per inch per deg. Cent. change in temperature, and 0.00013 in. per inch per deg. Cent. in the "B" direction.

It was found that the rubber increased in weight 0.05 per cent when submerged in water for 7 days. The change in length incident to this absorption was found to be within the limits of experimental error, and therefore may be considered as negligible if not zero.

## CONCLUSIONS

The results obtained using the methods described, appear to indicate that for the lower stress ranges very satisfactory physical property values can be determined for a wide variety of rubber compounds. Satisfactory values for the modulus of elasticity in tension, compression and flexure; Poisson's ratio; permanent set; elastic hysteresis; creep; specific gravity; thermal expansion; and water absorption were obtained.

The data presented show some very interesting properties for at least one of the rubber compounds investigated.

*Acknowledgment.*—The authors wish to express their appreciation of the assistance given by Messrs. R. L. Moore and F. E. Rebhun who made many of the tests in the investigation.

# SYMPOSIUM ON ABRASION TESTING OF RUBBER

## INTRODUCTION

By Harlan A. Depew[1]

This symposium on the abrasion testing of rubber may be considered as a decennial celebration of the work on abrasion testing of the Society's Committee D-11 on Rubber Products. The first part of the decade was formative during which time Committee D-11 was groping for some means that would make their work serve the interest of both producers and consumers more satisfactorily. It was decided that this could be achieved best by devoting more time to the development of performance tests and in standardizing these tests. Accordingly, a subcommittee on performance tests was appointed in 1923, and under the able chairmanship of C. R. Boggs it grew so rapidly that in 1927 it had to be divided into three parts: aging, flexing, and abrasion.

The early activities of the abrasion group while still a part of the performance test committee consisted in an investigation of the ability of laboratories to obtain check results using existing abrasion machines, with the result that abrasion testing was found to be as reliable as tension testing.[2] The next activity was an extensive comparison of service and laboratory tests, which showed good correlation under favorable circumstances.[3] The present activity of the Subcommittee on Abrasion has progressed in the direction of investigating the nature of abrasion, including the question of the relative importance of tear resistance and cutting resistance.

It is believed that a great deal of information has been collected on abrasion testing and one of the important reasons for arranging for this symposium has been to offer a place where this information could be brought together. Accordingly, the discussion will cover not only the papers presented, but also other subjects of interest in connection with abrasion testing of which the following subjects are offered as examples:

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[1] Chief, Rubber Section, Research Division, The New Jersey Zinc Co., Palmerton, Pa.; Chairman, Subcommittee XIV on Abrasion Tests for Rubber Probucts, A.S.T.M. Committee D-11 on Rubber Products.

[2] *Indian Rubber World*, Vol. 74, August 1, 1926, p. 266.

[3] *Indian Rubber World*, Vol. 81, No. 2, November 1, 1929, p. 66.

1. Notes on the correlation of service tests with laboratory tests;
2. Unusual service conditions and results;
3. The design of an ideal abrasion machine; and
4. Tearing and cutting tests and their value in determining abrasion resistance.

There can be no doubt but that confidence in abrasion testing has increased greatly in the last ten years, and it does not seem unreasonable, considering the controlled conditions of laboratory testing, to expect that in the next ten years laboratory abrasion testing will reach a place of confidence where it will be considered more reliable than the service test unless a very large number of the latter is used.

# STUDY OF A TEST FOR TEAR RESISTANCE OF VULCANIZED RUBBER COMPOUNDS

By A. W. Carpenter[1] and Z. E. Sargisson[2]

## Synopsis

This paper gives results of tearing tests using compounds of which the abrasion resistance has been investigated by Subcommittee XIV on Abrasion, of the Society's Committee D-11 on Rubber Products. Comparison is made between tear resistance, abrasion resistance and stress-strain tests. The tear test procedure and the required equipment are described. The method is not new but has not previously been described in detail. Effect of variations in compounding, mechanical handling and cure of the rubber stocks is dicussed.

The Goodrich tear test for vulcanized rubber has been in use for a number of years and is variously known as the Winkelmann tear test, the crescent tear test and popularly as the peanut tear test. It was originally designed by H. D. Ayres, and has since been used by numerous investigators and has been modified in several details. The principal modifications have been in respect to the shape of the test specimens and the equipment used in preparing them.

The purpose of this paper is to present a study of some factors involved in tear resistance as measured by this test. The method of test is described in detail. A recent design of the test specimen preparation apparatus together with information on the effect of a change in shape of the specimen are given. The data presented show the effect on tear resistance produced by some variations in the compounding and mechanical handling of rubber stocks prior to vulcanization. Since resistance to tear is considered to be one factor which is involved in abrasive wear, the compounds chosen for some of the work were those previously used by Subcommittee XIV on Abrasion, of the Society's Committee D-11 on Rubber Products, and abrasion test data are included for comparison.

## Equipment and Method of Test

The method of test consists in subjecting a crescent-shaped test specimen to a tensile load under conditions to produce tearing. A standard power-driven tension testing machine of the pendulum type is used. The specimens are die cut from vulcanized test sheets which are approximately 0.1 in. in thickness and the actual gages are measured. After five small slits are made in the inner curved surface of each specimen, it is placed in the machine and loaded to rupture. The load required to separate the specimen is used in calculating the resistance to tear.

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[1] Manager of Testing Laboratories, The B. F. Goodrich Co., Akron, Ohio.

[2] Testing Engineer, The B. F. Goodrich Co., Akron, Ohio.

The shapes of specimens ordinarily used at present are shown in Fig. 1. The curved shape and the small nicks were apparently designed to insure an excess stretch on one edge and to provide for selective tearing. Originally the curved edges had decidedly smaller radii than those shown, but experience has resulted in the use of less sharp curves to give a smaller differential in stretch between the two sides. It will be noted that the modified shape has slightly less curvature than the Goodrich Standard.

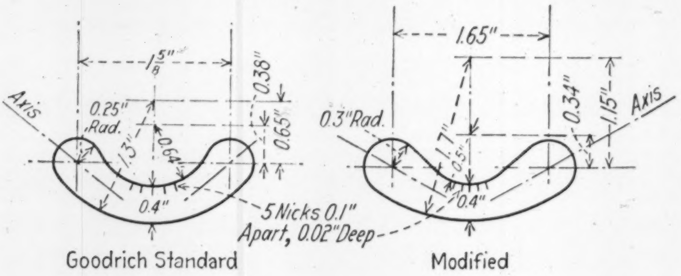

FIG. 1.—Showing Tension-Test Specimens.

Modified shape used with tear-test equipment supplied by the Akron Equipment Co., Akron, Ohio.

The small slits in the specimens must be cut with extreme care and accuracy. To realize proper selectivity in tear, they must all extend to a uniform depth in the rubber. Before the tear starts during test, a specimen will have been stretched and the curved edges drawn out straight. The slits must then be parallel and at right angles to the edges. To secure these conditions, equipment for use in preparing the specimens has been designed as shown in Fig. 2. A special device which holds the specimens stretched so that the inner curved edges are rectilinear is provided with suitable guides to control depth and centering of the slits. The stretching of the specimens in this device aids in securing clean and accurately centered slits. The cutting is done by safety razor blades clamped in a holder so as to be spaced exactly 0.1 in. apart. The extension of the blades beyond the surface of the knife holder is regulated by a depth gage block so that when used with the specimen device, the blades give slits all extending to a depth of 0.02 in. In cutting the slits, the specimens held in the stretching device are first painted with soap solution and the knives are then drawn back and forth through the guides.

In clamping a specimen for test in the jaws of the testing machine, care must be taken that the jaws grip the specimen so that the axes shown in Fig. 1 are in line with the direction of the application of the load. Increased stretch is thus given to the inner curved edge. The "bite" of the jaws should be at the center of the enlarged ends of the specimen. In order to insure proper clamping, it is desirable to use narrow cam or pincer type jaws. The load is applied with a lower jaw speed of 20 in. per minute.

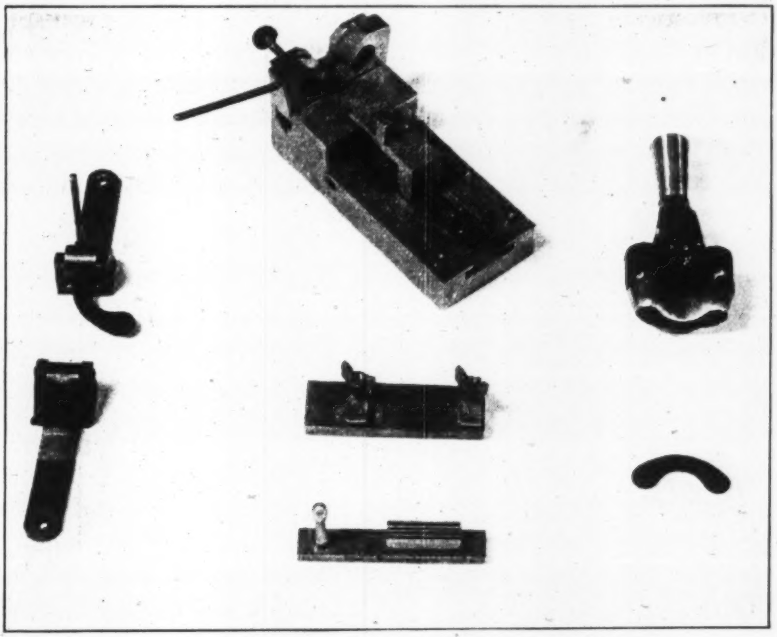

FIG. 2.—Showing Equipment for Use in Preparing Specimens.

The resistance to tear is calculated from the maximum load registered by the testing machine and the actual thickness of the specimen, and is taken as the pounds pull which would have been required to separate the specimen if it had been exactly 0.1 in. in thickness. Resistance to tear, therefore, is equal to the maximum load in pounds divided by ten times the actual thickness in inches. For example, if a specimen having a thickness of 0.092 in. pulls 46 lb., the resistance to tear would then be:

$$\frac{46.0}{10 \times 0.092} = 50.0 \text{ lb.}$$

## EXPERIMENTAL DATA

In this study, the aim was (1) to ascertain the extent to which the resistance to tear is affected by the grain of rubber compounds; (2) to determine the effect of different states of cure on tear resistance; (3) to investigate the effect of changing the curvature of the test specimen in so far as the two shapes illustrated are concerned; (4) to compare the resistances to tear secured by various types of compounding with each other and with results of stress-strain and resistance to abrasion tests.

In carrying out this program, seven rubber compounds were used, the recipes for which are as follows:

| Compound | Composition, parts by weight | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | A.S.T.M. No.[a] | Pale Crepe | Smoked Sheet | Diorthotolyl Guanidine | Sulfur | XX Red Zinc Oxide | Micronex | Double Pressed Stearic Acid | Medium Pine Tar | Whole Black Tire Reclaim | Hexa |
| A—Low-Carbon Black Tread | 5 | 50 | 50 | 1.25 | 3.5 | 18.2 | 24.4 | 2 | 2 | .. | .. |
| B—High-Carbon Black Tread | 6 | 50 | 50 | 1.25 | 3.5 | 18.2 | 40.0 | 2 | 2 | .. | .. |
| C—Second-Grade Tread | 8 | 40 | 40 | 1.12 | 3.5 | 18.2 | 37.5 | 2 | 2 | 33 | .. |
| D—Third-Grade Tread | 9 | 30 | 30 | 1.0 | 3.5 | 18.2 | 35 | 2 | 2 | 66 | .. |
| E—Under-Accelerated Tread | 10 | 50 | 50 | 0.45 | 3.5 | 18.2 | 40 | 2 | 2 | .. | .. |
| F—High-Zinc Standard[b] | .. | .. | 100 | .. | 6 | 137 | .. | .. | .. | .. | 1 |
| G—High-Gum Standard | .. | .. | 100 | .. | 6 | 5 | .. | .. | .. | .. | 1 |

[a] *India Rubber World*, Vol. 81, No. 2, November 1, 1929, p. 66.

[b] H. A. Depew, "Abrasion Testing of Rubber," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 873 (1928).

The batches were mixed on a 12 by 24-in. laboratory mill having a slow-roll speed of 14 r.p.m. and a gear ratio of 1.71. A uniform mixing procedure was used in all cases and great care was taken after the ingedients were incorporated in the rubber to control the grain effect as desired. A large batch of each compound was divided, after having been mixed and cooled. One-half of the batch was then broken down and sheeted directly from the mill, using the procedure recommended by the Physical Testing Committee of the American Chemical Society, Rubber Division. By this method, the stock was passed through the mill six times after it was well broken down, and each time the direction of feeding the stock was changed to cross the grain. After the sixth pass, the stock was allowed to adhere to the front roll for two or three revolutions to form a smooth sheet and immediately removed from the mill. This is designed to destroy the grain as much as possible. The other half of each batch was warmed up and passed six times through the mill with the feed always in the same direction of the grain. This half was then sheeted using an 8 by 14-in. even-speed laboratory calender. In supplying the calender, the stock was cut from the mill and fed into the calender with the grain in the same direction as on the mill. This was done to emphasize the grain effect. The sheeting operations were carried out so as to give thickness of raw stock which produced minimum flow in the molding operations.

Test sheets 6 by 8 by $\frac{3}{32}$ in. were vulcanized in molds using a platen press. The raw stock for these sheets was cut and placed in the mold cavities so that the 8-in. dimension in all cases extended in the same direction as that of removal of the stock from the rolls and the quantity was regulated to give very slight overflow. The temperature of cure was automatically controlled and the platens were calibrated for impressed temperature using thermocouples. The duration of the cure was accurately checked and at its conclusion the vulcanized sheets were immediately removed and cooled in water.

After the test sheets had aged a minimum of 48 hours, specimens for tear and stress-strain tests were cut from the same sheets. For the stress-strain dumb-bells, the standard American Chemical Society die was used and they were cut longitudinally of the sheets. The crescent-shaped tear test specimens were cut with the long dimension of the dies placed in both directions on the sheets. The specimens for which the long dimension of the die was lengthwise of the sheet had the grain, if any, running lengthwise of the specimen. The direction of tear during test was then started across the grain. These are, therefore, referred to as transverse tests. Conversely, the specimens cut across the test sheet gave longitudinal tear tests. The tear tests were carried out on an Olsen rubber tension testing machine while for the stress-strain tests a Scott rubber tester with compensating head attachments was used.

The results of the tests using all seven compounds at five different cures are given in graphical form. With all test sheets, tear tests were made in both longitudinal and transverse directions and both the Goodrich and modified dies were used. Each point on the curves showing these data is the average of determinations on five specimens. In the case of the stress-strain results, three specimens of each were tested and the averages of the two showing the highest tensile strength were used providing they checked each other within 10 per cent. Otherwise, additional specimens were run.

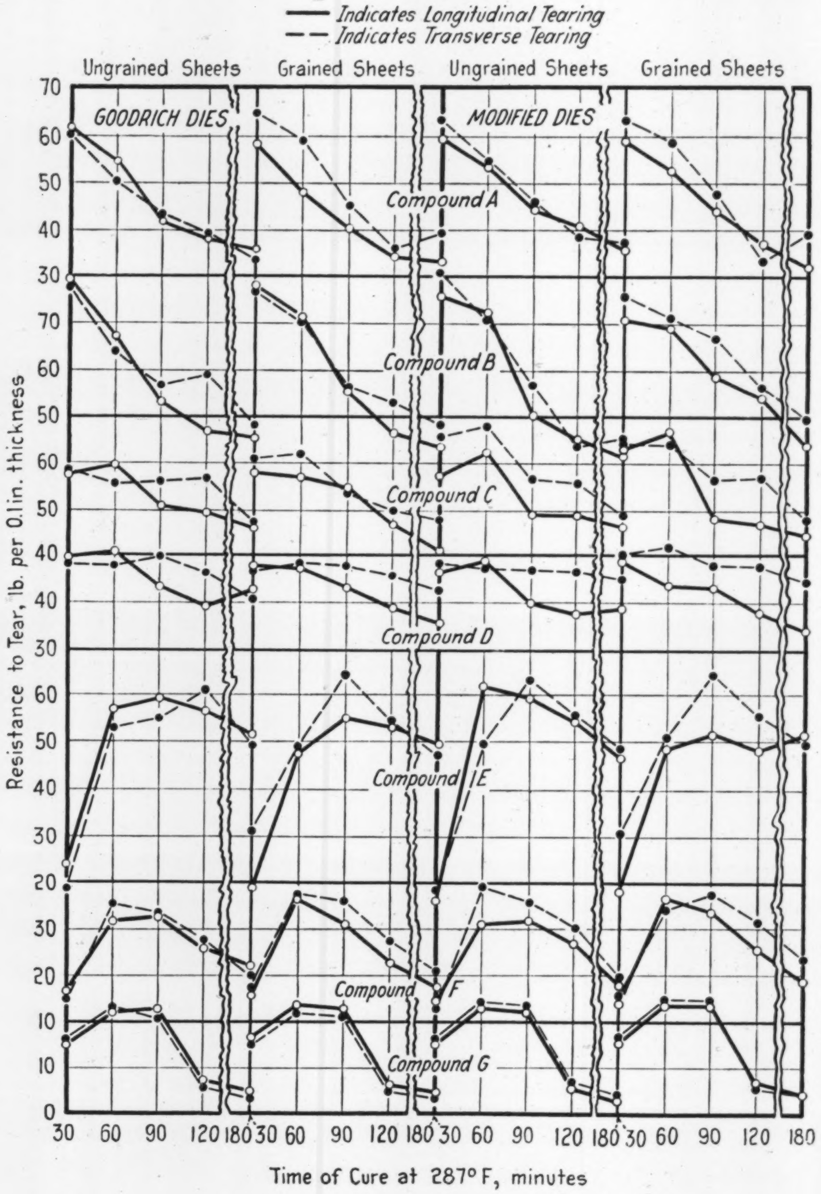

FIG. 3—Showing Tear-Test Values for All Conditions of Time and Temperature of Cure Used with the Compounds Studied.

## DISCUSSION OF RESULTS

### *Influence of "Grain-Effect" in the Tear Test:*

In studying the influence of grain-effect on the resistance to tear, it was expected that marked differences would be found between the values secured when tearing longitudinally as compared with the transverse tests, particularly with the test sheets which were carefully grained and where existence of the grain was definitely shown by observation through hand tears. It was thought that these differences should not be so pronounced when the sheets were not grained as in American Chemical Society mixing procedure. This was not found to be the case. In Fig. 3, it will be noted that the graphs for each compound showing data taken in both directions of tear for each method of mixing are superimposed so that the effect of the grain can be readily observed. It is apparent that no consistent differences are shown in the tear values which could be attributed to the grain. In some instances, the transverse tear is slightly greater than the longitudinal and in others the reverse is true. Also, the transverse results with the grained sheets in general are no higher than those with the ungrained sheets. Evidently, the differences shown are due

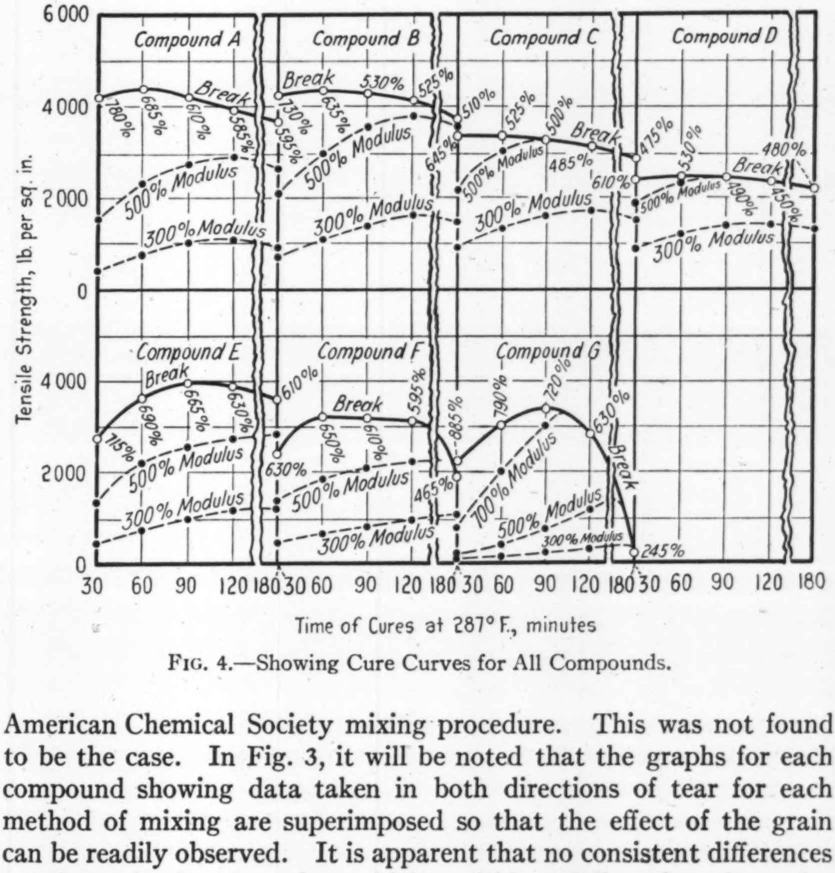

FIG. 4.—Showing Cure Curves for All Compounds.

to inaccuracies of the method, experimental errors or causes other than grain-effect.

During these tests, the observation was made that with 1400 specimens, the tearing started in one or the other of the two end nicks in 75.6 per cent of the tests. Sixty-eight per cent of the longitudinal tears and 83.5 per cent of the transverse tears were in this class. It was further noted that in the transverse tearing, the direction was not directly across the specimen but tended to follow the longitudinal direction of the grain.

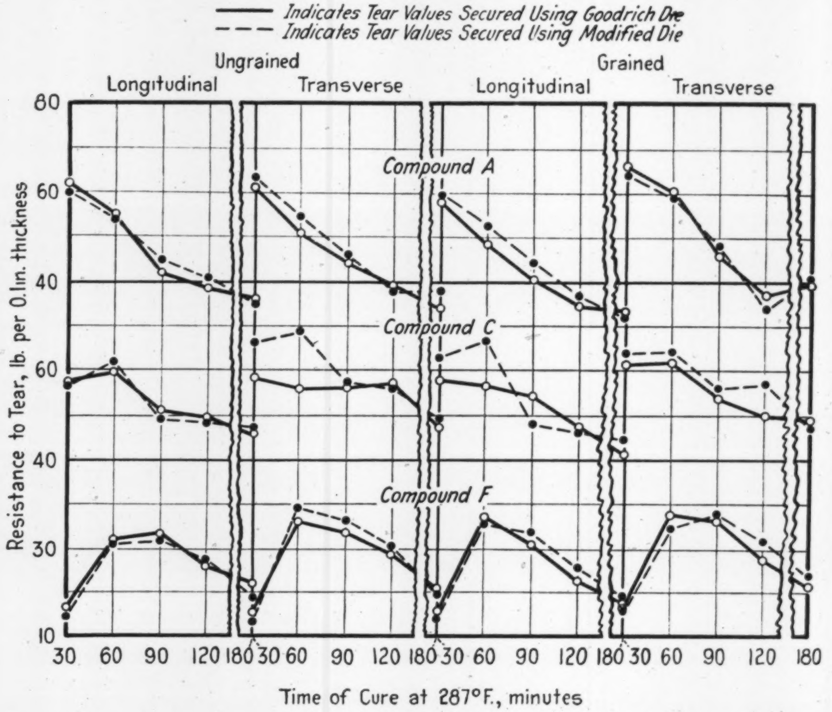

FIG. 5.—Showing Tear-Test Values of Compounds A, C, and F, Using the Goodrich and Modified Dies.

*Effect of State of Cure:*

Figure 3 presents the tear test values for all of the conditions of time and temperature of cure used with the compounds studied. In order to confirm the states of cure represented in these graphs, stress-strain cure curves are given in Fig. 4. It is to be noted that all of the specimens have progressively lower resistance to tear as the amount of overcure is increased. Of the compounds tested, it is seen that with the shortest time used, stocks A, B, C and D are at or near the optimum cure, as indicated both by the tear resistance and tensile strength curves. With stocks E, F and G, the 30-minute cures are distinctly under the optimum according to the stress-strain data. These undercures are also seen to have reduced resistance to tear.

*Effect of Specimen Contour:*

For comparison between the two shapes of the specimens, the data previously referred to have been re-plotted in Fig. 5 for three of the compounds which represent different types. These graphs show general agreement between the tests made with the two shapes. Such differences as do exist are again in both directions and force the belief that they are due to experimental error, inherent lack of precision of the method or causes other than the differences in shape.

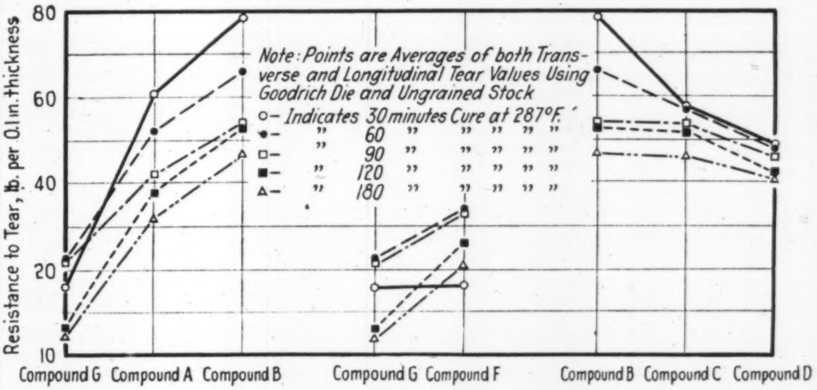

FIG. 6.—Showing Changes Due to Compound Variations.

*Effect of Compound Variations:*

If the tear test results shown in Fig. 3 for compounds G, A and B are compared, it is evident that the use of carbon black within the range covered is an effective means of increasing the resistance to tear. Stock G, which is a high rubber compound, gave 23 lb. for the tear value of the 60-minute cure. Stock A, the low-carbon-black tread, showed an increase in tear resistance to 53 lb. or approximately 130 per cent while the high-carbon-black tread B gave an increase over the rubber compound to 66 lb. or about 187 per cent. These increases cannot with fairness be entirely attributed to the carbon black because of other differences in the compounds such as accelerator, sulfur ratios and use of additional ingredients. However, the improvement of stock B over stock A is entirely due to the added carbon black.

By comparing compounds B, C and D, the effect of adding reclaimed rubber is brought out. The first-grade tread gave 79 lb., the second-grade 57 lb., and the third-grade 48 lb. for the tear values of the 30-minute cures.

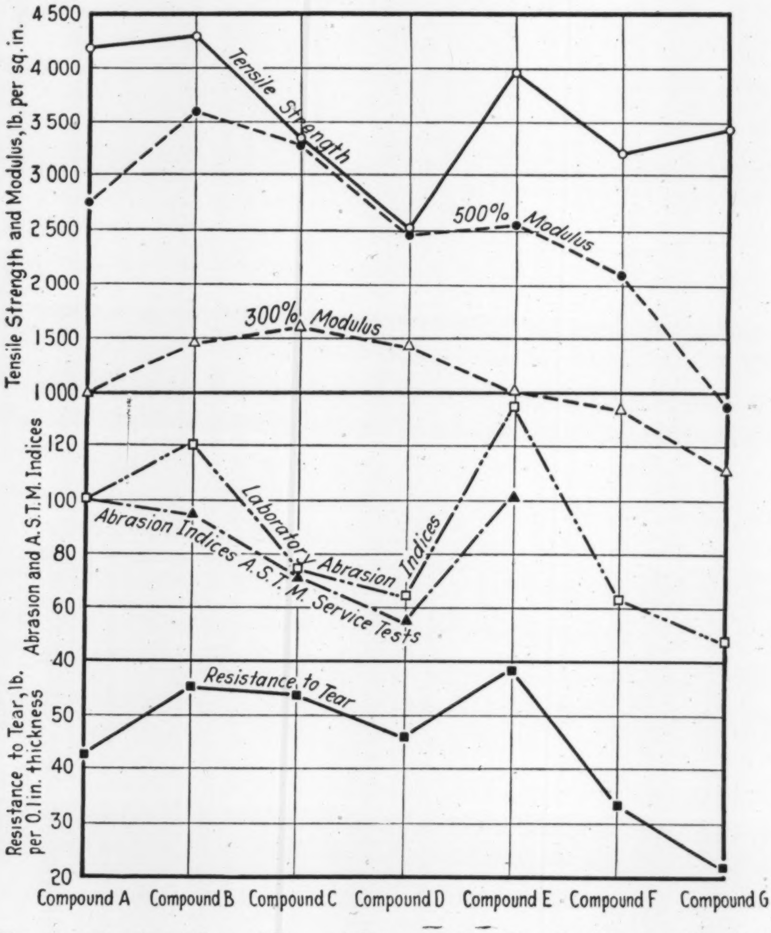

FIG. 7.—Showing Effect of Type of Compound on Tear Resistance, Abrasion Resistance, and Stress-Strain Characteristics, 90-minute cure.

Compounds F and G show the effect on tear resistance of adding zinc oxide. When 20 volumes of XX red zinc oxide were added per 100 volumes of rubber, the tear value was increased from 22 lb. to 34 lb. or approximately 55 per cent in the case of the 60-minute cures. These comparisons are shown in Fig. 6.

*Comparison of Tear Value with Abrasion Indices and Stress-Strain Data:*

Figure 7 shows the comparison between tear value, abrasion indices and stress-strain data. The stress-strain, tensile strength and tear test data shown were taken from the sheets cured 90 minutes at 287° F. The abrasion indices are those given by Subcommittee XIV of Committee D-11, and represent the results of actual tire tests. The laboratory abrasion indices[1] were taken from tests on an angle machine abrading ring specimens cured 90 minutes at 287° F. It appears from this comparison that the tear values for the different types of stocks show a distinct tendency to vary in the same directions as the abrasion indices and as the stress-strain and tensile strength data.

## CONCLUSIONS

1. Resistance to tear as determined by the Goodrich tear test does not measure differences due to "grain-effect." It is believed that the explanation of this lies in the observation that the actual tearing is always along the grain under the conditions of test. The tearing force measured is, therefore, that required to separate the fibrous structure of the rubber rather than that necessary to break the fibers themselves. It may even be possible that in tensile strength tests, the actual rupture is mainly between the fibers and around their ends rather than across them.
2. The resistance to tear measured by this test is a function of state of cure, being greatest at or near the optimum cure and lowered both by undercure and overcure.
3. A small change in the curvature of the specimen does not appreciably affect the results of the determination.
4. Addition of carbon black or zinc oxide to the rubber compounds and within the proportions investigated increases the resistance to tear.
5. Increasing amounts of the typical whole black tire reclaimed rubber used in the tread compounds studied, progressively lowered the resistance to tear.
6. The resistance to tear of a given compound as measured by this test appears to be related to the tensile strength, the modulus and the resistance to abrasion of the compound.

*Acknowledgment:*—The authors wish to express their appreciation for the assistance rendered by Mr. R. C. Cook, especially in the preparation of the compounds and test sheets.

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[1] J. L. Tronson and A. W. Carpenter, "Abrasion Tests of Vulcanized Rubber Compounds Using an Angle Abrasion Machine," see p. 908.

# ABRASION TESTS OF VULCANIZED RUBBER COMPOUNDS USING AN ANGLE ABRASION MACHINE

By J. L. Tronson[1] and A. W. Carpenter[2]

## Synopsis

This paper describes the machine used and the method of test in the abrasion testing of vulcanized rubber compounds. Data are presented on the effect of variations in load, speed, temperature and size of test specimen. The state of cure and effect of accelerated aging is also considered. Comparisons are made with road tests on several tire tread compounds.

The object of this work was to devise an abrasion machine which would give results on tread compounds which are comparable to results obtained by actual road tests. It had been shown by Vogt at Goodyear[3] and Lambourn at Dunlop[4] that abrasion machines of the angle-slip or constant-slip type give results which evaluate service fairly well.[5] As the Goodyear machine is not complicated and quite easy to build, a machine similar to it was constructed.

The tests described in this paper were made with the idea of checking up on the performance of the machine under various operating conditions. Comparisons are given with road test data. Test results are also presented showing the effect on abrasion loss of various methods of processing the stocks previous to test.

## Description of the Machine

The machine used in these tests is shown in Fig. 1.

The specimen is in the form of a ring, 2 in. in outside diameter and 1 in. in inside diameter and is mounted on a holder set on the shaft of a $\frac{1}{6}$-h-p. alternating-current motor. The abrasive consists of an aloxite wheel, grading 60-J-D-842, 12 in. in diameter and $1\frac{1}{2}$ in. thick set on a horizontal shaft driven by a $\frac{1}{4}$-h-p. alternating-current motor. Both motors are not used simultaneously but were included in the design to provide flexibility so that either specimen or abrasive could be driven. The center line of the shaft holding the specimen is set 1.25 in. above the center line of the abrasive shaft so that there is an angle of 13 deg. between the directions of motion of the specimen and the abrasive at the point of contact. The motor holding the specimen is set in grooves so that by means of a system of pulleys, a dead weight load can be applied horizontally to it, thereby forcing the test ring against the abrasive with the desired pressure.

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[1] Development Engineer, The B. F. Goodrich Co., Akron, Ohio.

[2] Manager of Testing Laboratories, The B. F. Goodrich Co., Akron, Ohio.

[3] W. W. Vogt, *Industrial and Engineering Chemistry*, Vol. 20, p. 302 (1928).

[4] L. J. Lambourn, *Transactions*, Inst. Rubber Industry, Vol. 4, p. 210 (1928).

[5] C. W. Sanderson, *India Rubber World*, Vol. 80, No. 4, p. 53 (1929).

A revolution counter was used to record the revolutions made by the abrasive during test.

The specimen holder is carried on the motor shaft by means of a steel bushing to which it is keyed so that no slippage can take place. The diameter of the holder is $\frac{1}{16}$ in. greater than the inside diameter of the test ring. The test ring is held on the holder by means of a disk fastened in place by a clamping device and kept in position by a steel spring.

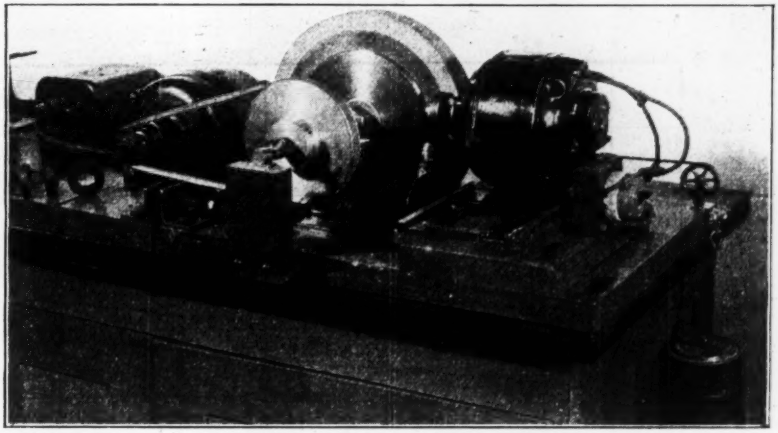

FIG. 1.—Showing Abrasion Machine used in Tests.

A brush is set against the test ring to remove the abraded rubber. With properly cured tread compounds, there is little tendency for the abraded rubber to cling to the abrasive during test with this type of machine.

## METHOD OF TEST

A large portion of this work was carried out with the idea of checking up on various operating conditions. In addition to tests run to show the effect of speed, specimen thickness, temperature, load on the specimen, and time of cure, two sizes of test rings and three types of holders were tried out. Some tests were run by driving the specimen while having the motor on the abrasive disconnected. When tests were run with the abrasive driven, the motor to which the specimen was attached was used as a holder only. For some of the work, the alternating-current motor driving the abrasive was replaced by a variable speed direct-current motor so that tests at various abrasive speeds could be made.

The following procedure is being used at present based on the results of numerous tests:

1. Test rings ½ in. wide are used. These should be given a slight overcure to avoid the erratic results obtained on undercured stocks. The rings used in any series of comparative tests should be buffed to equal widths within ± 0.001 in. before test.

2. A load of 18 lb. is used to hold the test ring against the abrasive.

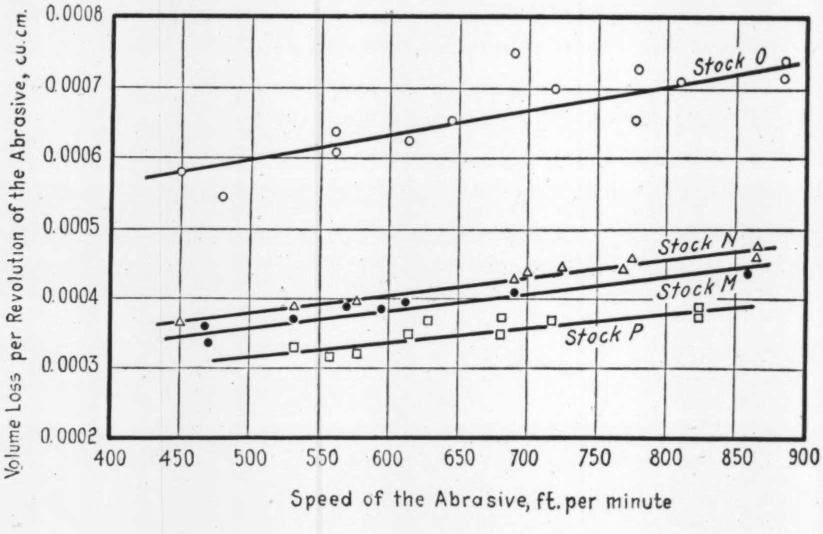

FIG. 2.—Showing Effect of Speed on Abrasion Resistance.
⅜-in. rings; abrasive driven; load on sample, 16 lb.

3. The test is run with the abrasive driven at a speed of 485 r.p.m. This is equivalent to a linear speed at the point of contact of the abrasive and the specimen of 1450 ft. per minute.

4. The tests are run to give approximately equal volume losses rather than for a given number of revolutions. The method used at present is to continue abrading the test specimen until the outside diameter of the ring is 1.70 in. ± 0.01 in. at which time the test run is concluded and the number of revolutions of the abrasive is recorded.

5. Results are reported as volume loss per 1000 revolutions of the abrasive. The volume loss is calculated from the loss in weight and the specific gravity of the specimen.

6. The room temperature should be uniform over the period in which a series of tests is made. A variation of 2 to 3° F., however, is not important.

7. All tests are run as comparisons between two or more stocks. If possible, tests are repeated on successive days. Changes in room temperature, particularly in summer months, as well as other uncontrollable variables, make it necessary to run in close succession all stocks which are to be compared. The comparisons are made on a volume loss basis rather than in terms of indices of abrasion.

## Stocks Tested

A large number of tread stocks were tested on this machine. As a rule, they were run in groups which had been tested together on the road. The stocks used for most of this work are as follows:

| Group | Compound Designation | Description |
|---|---|---|
| I | A No. 5 | Compounds chosen by Society's Committee D-11 on Rubber Products, Subcommittee XIV on Abrasion, and used in a study of abrasion testing.[a] |
| | B No. 6 | |
| | C No. 8 | |
| | D No. 9 | |
| | E No. 10 | |
| | F | High zinc-oxide standard used in work published by H. A. Depew.[b] |
| | G | High Gum Stock—100 smoked sheets, 5 XX red zinc oxide, 6 sulfur, 1 Hexa, parts by weight. |
| II | H | First-Grade Tread—28 parts carbon black per 100 parts rubber[c] |
| | I | First-Grade Tread—25 parts carbon black per 100 parts rubber[c] |
| | J | First-Grade Tread—23 parts carbon black per 100 parts rubber[c] |
| | K | Second-Grade Tread—28 parts carbon black per 100 parts rubber[c] |
| | L | Second-Grade Tread—23 parts carbon black per 100 parts rubber[c] |
| III | M | High-Grade Tread—6 per cent black tire reclaim, 22 parts black per 100 parts rubber[d] |
| | N | Second-Grade Tread—12 per cent black tire reclaim, 24 parts black per 100 parts rubber[d] |
| | O | Third-Grade Tread—32 per cent black tire reclaim, 24 parts black per 100 parts rubber[d] |
| | P | First-Grade Tread—No reclaim, 22 parts black per 100 parts rubber[d] |
| IV | Q | Similar stocks—differing only in the kind of accelerator and sulfur ratio used. |
| | R | |
| | S | |
| | T | Same as stock Q, but containing 10 per cent whole tire reclaim |

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[a] *India Rubber World*, Vol. 81, No. 2, p. 66, November 1, 1929.

[b] H. A. Depew, "An Explanation of Some of the Difficulties in Abrasion Testing of Rubber," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 871 (1928).

[c] Ratios by weight of total rubber for zinc oxide, sulfur accelerator and stearic acid are the same in all cases, except stock I has the acid reduced one-half. Compounds K and L contain 12 per cent black tire reclaim.

[d] The ratios by weight on total rubber for zinc oxide, sulfur accelerator and stearic acid are the same for stocks M and N. Stock O is similar to M and N with lower accelerator and sulfur ratios. The accelerator used in stock P is different from that used in the other three stocks.

## DISCUSSION OF RESULTS

Some preliminary work was done on a modification of this machine which consisted in placing a prony brake arrangement on the abrasive shaft, setting the specimen in such a position that there was no angle between it and the abrasive at the point of contact, and driving the abrasive by the motor which holds the specimen. Tests run on a series of stocks against constant prony brake loads showed that a considerable difference existed in the "slip" taken by different stocks as determined by average surface linear speeds of abrasive and specimen during the tests. Low-modulus stocks showed a much greater tendency to slip than high-modulus stocks, with much faster wear regardless of quality of the stock as shown by road tests. However, when the results were compared on a constant slip basis, they lined

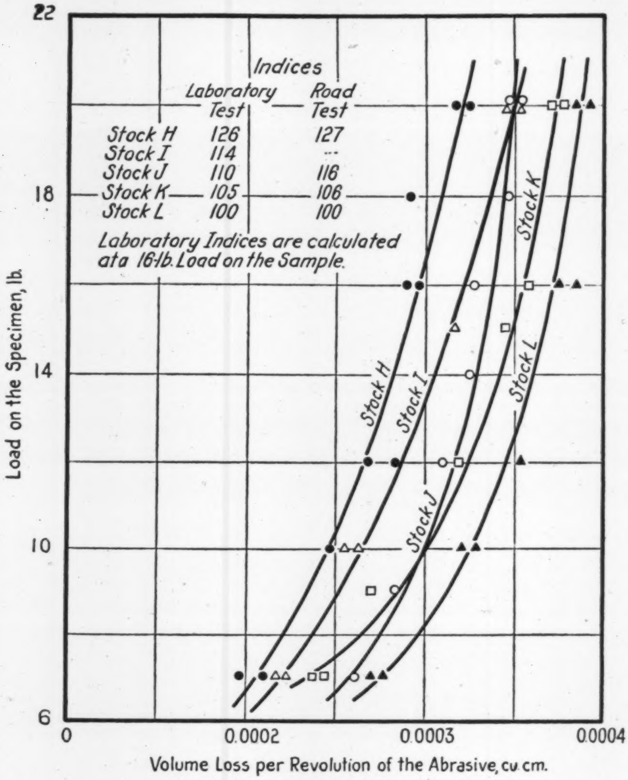

FIG. 3.—Showing Effect of Variation in Load on Abrasion Resistance.
$\frac{3}{8}$-in. rings; sample driven; speed, 625 ft. per minute.

up fairly well with the road tests. Work with the prony brake was discontinued because of the difficulty in adjusting the brake to give check results and because constant slip could be more easily obtained by setting the specimen at an angle with the abrasive.

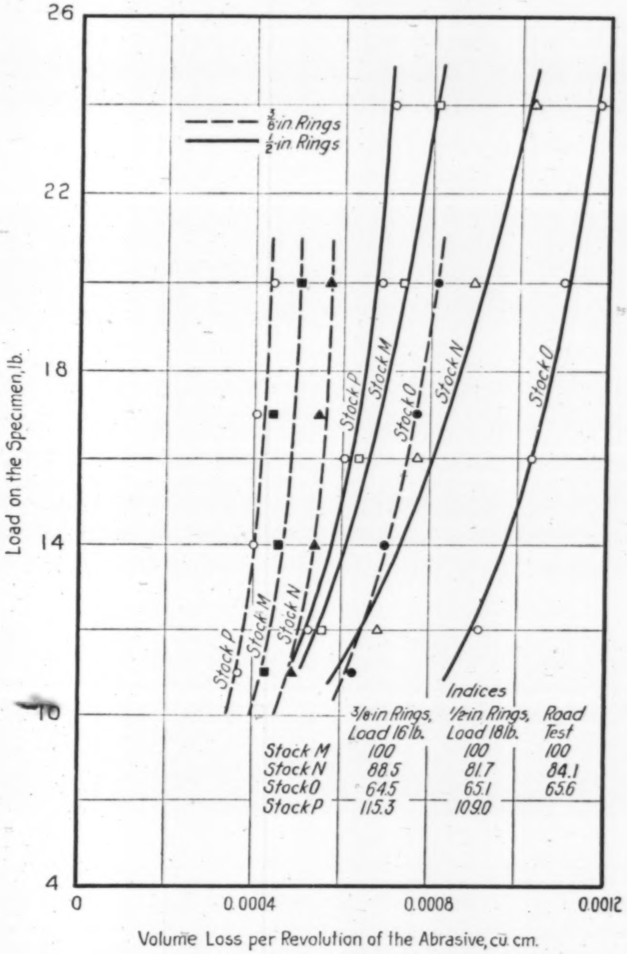

FIG. 4.—Showing Effect of Variation in Load on Abrasive Resistance.
**Abrasive driven; speed, 840 ft. per minute.**

*Effect of Variation in Width of the Test Ring:*

A variation of 0.01 in. in the width of $\frac{3}{8}$-in. test rings was found to cause errors as high as 14 per cent. For most of this work the rings were buffed to an equal width for every series of tests. This item must be watched carefully, as slight differences in the amount of uncured stock placed in the mold will cause noticeable differences in the width of the cured rings. Equal volumes of different stocks will give cured rings of different width, due to variation in molding conditions.

### *Comparison of Test Rings $\frac{3}{8}$ in. and $\frac{1}{2}$ in. Wide:*

The use of rings of either $\frac{3}{8}$ in. or $\frac{1}{2}$ in. width for a series of tests will give the same comparative results. Three-eighths-inch rings were used for some of this work. However, it has been found that $\frac{1}{2}$-in. rings tend to give more uniform results, possibly due to the higher volume loss per unit time of test, and to less influence from uncontrolled variations in conditions of test. In testing rings of different width, the loads with which the samples are held against the abrasive should be proportional to give uniform intensity of pressure. The results of tests with both sizes of rings under different load conditions are given in Fig. 4.

### *Comparisons of Holders:*

The preliminary tests were run using a holder on which the specimen was held by a knurled washer and lock-nut arrangement. While this was satisfactory so far as the results were concerned, the present clamp type of holder is more convenient to handle. The holder must be rigid and set up so that all of the specimens are held in the same position during test. While running, the specimen is forced against the outside clamp by the rotation of the abrasive which may tend to disturb its proper position. The steel spring is used to keep fairly uniform pressure against the sides of the specimen.

### *Abrading to a Constant Volume Loss versus Running for a Given Number of Revolutions:*

The rate of wear is not constant during a given test but varies as the diameter of the test ring changes. At first, the rate of wear is relatively rapid, owing to removal of the corners and shaping of the specimen to the abrasive surface. Then it drops to a minimum and again rises as the specimen diameter approaches that of the holder. It appears that the amount of rubber behind the surface being abraded, which is free to give cushioning effect and to allow bending of the specimen, has a distinct influence on rate of abrasion. Possibly this is involved in the fact that with some laboratory machines, when testing abrasive wear of compounds of different degrees of hardness, the hard stocks may show superior results compared with the softer stocks even though the service results may indicate that the opposite should be true. Such machines may use test specimens too thin to permit proper influence of the factor of modulus of the stock in the test conditions. Further work is being done to investigate these points. With the machine described, it has been found better to abrade all rings to a given diameter rather than for a given number of revolutions. For stocks which differ by ten per cent or less in abrasive resistance, the latter procedure was satisfactory, but when the difference in stocks was very great, the former method gave much more reliable results.

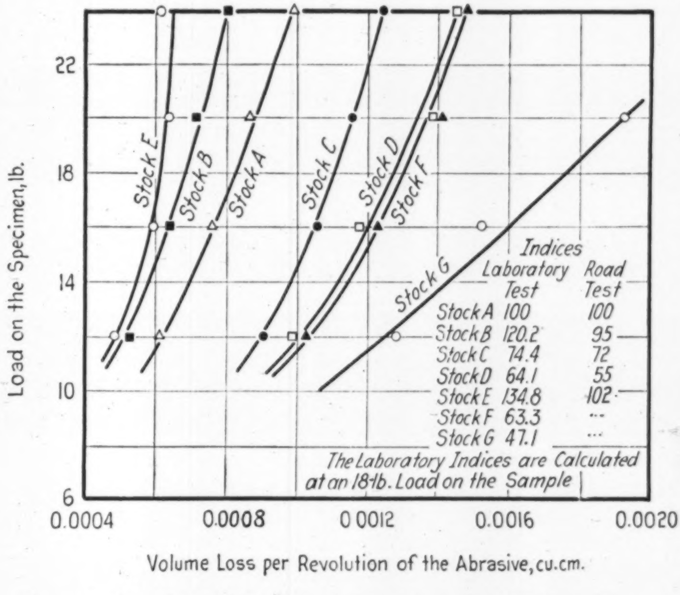

FIG. 5.—Showing Effect of Variation in Load on Volume Loss.
**$\frac{1}{2}$-in. rings; abrasive driven; speed, 1450 ft. per minute; cure, 90 minutes at 40 lb.**

### *Specimen-Driven versus Abrasive-Driven Tests:*

The same relative results are obtained with the specimen driven as when the abrasive is driven. However, when the specimen is driven, the linear speed at the point of contact of the specimen and abrasive drops as the specimen diameter decreases, resulting in a variable which, while practically constant over a series of tests, may be avoided by driving the abrasive. Also, the abrasive may be driven so as to give higher linear speeds at the point of contact than is convenient when the specimen is driven, due to the difference in active diameters and to the speed limitations of the electric motors.

*Effect of Speed:*

It was found that comparable results could be obtained with the speed of the abrasive varied over a considerable range, provided that the speed was held constant for any series of comparative tests. Tests have been run using linear speeds at the point of contact of abrasive and specimen varying from 450 to 1450 ft. per minute without noticeable change in the relative wear between stocks at any given speed. Most of these tests were run at speeds of 625, 840 or 1450 ft. per minute. Figure 2 shows the effect of speed on the volume losses of four stocks. These tests were run by driving the abrasive with a variable-speed direct-current motor. The speed range was from 430 to 880 ft. per minute, this being the maximum variation obtainable with the motor and pulley ratio used. The curves showing relation between speed and volume loss for the four stocks are practically parallel over the range in which the tests were made.

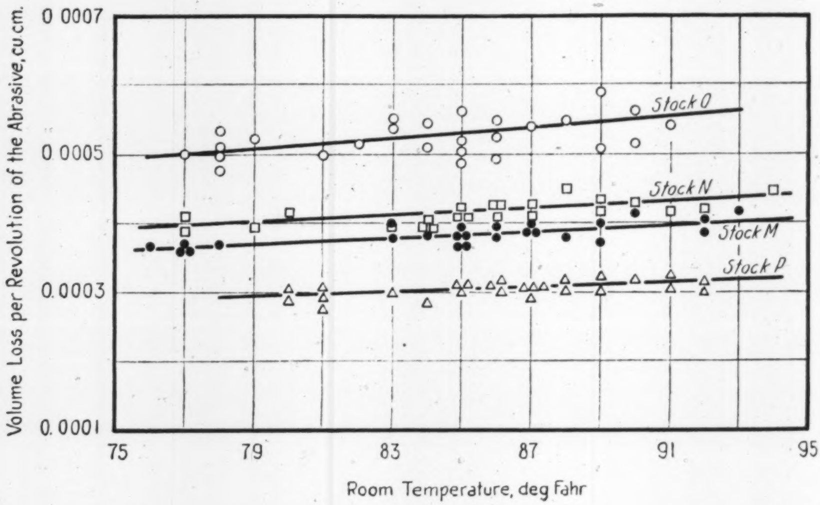

FIG. 6.—Showing Effect of Room Temperature on Abrasion Resistance.
⅜-in. rings; sample driven; speed, 625 ft. per minute; load, 16 lb.

*Effect of Load on Volume Loss:*

One of the most important factors studied was the dead-weight load applied to the specimen under test. Figures 3, 4 and 5 show the curves of volume loss *versus* load for three groups of stocks. In most cases, there is a tendency for the spread between the curves to increase as the load is made greater. That is, the differences in volume loss between stocks increase with the load. However, in the case of stocks I, J, and K (Fig. 3) this is not true. With all stocks tested, relative indices calculated from volume losses at a 16-lb. load for the $\frac{3}{8}$-in. rings and an 18-lb. load for the $\frac{1}{2}$-in. rings check fairly well with road indices. Both laboratory and road indices are recorded with the curves in Figs. 3, 4 and 5.

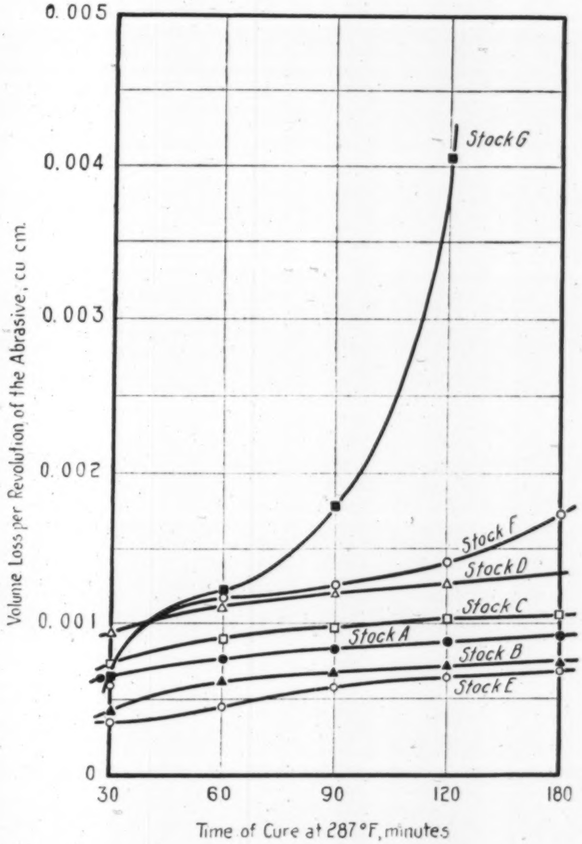

FIG. 7.—Showing Relation Between State of Cure and Abrasion Resistance. $\frac{1}{2}$-in. rings; abrasive driven; speed, 1450 ft. per minute; load, 18 lb.

*Effect of Temperature:*

Figure 6 shows the volume losses of four stocks plotted against the room temperature at the time the test was made. As these tests were made over a period of several weeks, there are other variables besides temperature affecting the results. Also, the temperature of the test specimen rises much higher during test than that of the room. However, these curves may give an indication of the change of volume loss with room temperature. Over a range of variation of 18° F. in room temperature, the volume loss was affected to the extent of approximately ten per cent, the greater loss occurring at the higher temperatures.

*Effect of Cure on Volume Loss:*

Figures 7 and 8 show the effect of degree of cure on volume loss for two groups of compounds. The points plotted represent average results of several tests run on different days. Figure 7 shows curves for the five tread stocks, A, B, C, D and E, used by Subcommittee XIV on Abrasion of the Society's Committee D-11 on Rubber Products as well as the zinc oxide compound used by Depew (stock F), and a high-gum stock (stock G) with a sulfur-accelerator-rubber ratio the same as in the zinc oxide standard. The five tread stocks give curves which are similar. The other two (stocks F and G) are quite different in character. The tread compounds were cured with diorthotolyl guanidine and the other two with Hexa.

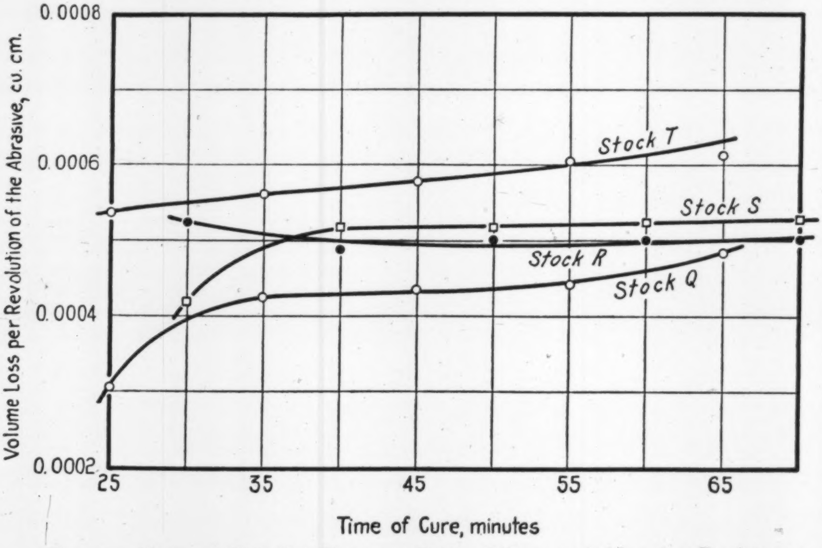

FIG. 8.—Showing Relation Between State of Cure and Abrasion Resistance. ½-in. rings; abrasive driven; speed, 1450 ft. per minute; load 18 lb.

In Fig. 7, it is seen that this test shows the superior abrasion-resisting qualities of the crude rubber tread compound containing high-carbon-black loading as compared with low-carbon-black ratio. Both of these are shown to be superior in proper order to the treads containing reclaim. The undercured tread shows the least abrasion loss of the group. This is in fair agreement with the test data published by Subcommittee XIV. The high-zinc-oxide stock and the unreinforced compound having different acceleration than the tread stocks are seen to be more sensitive to the state of cure. These last two compounds overcure badly and give very high abrasion losses with the overcured samples.

Figure 8 shows the relation between volume loss and time of cure for four tread stocks which contain 22 parts of carbon-black per 100 parts of rubber. These are designated Q, R, S and T. Stocks Q and T contain an aldehyde-amine accelerator. Stock R had mixed acceleration and in stock S, Captax was used. The rubber-black-zinc oxide-oil ratios of all these stocks are the same. The sulfur ratios and temperatures of cure used with the stocks were such that in the range of cures investigated, both undercured and slightly overcured rings were obtained as judged by behavior during abrasion, the undercures becoming soft and gummy during test. It can be seen from the shape of the curves that the change in abrasive resistance with cure depends a good deal on the accelerator used. Stocks T and Q are known to overcure more easily than stocks R and S. Here again, this is evidenced by the increasing abrasion loss with added cure.

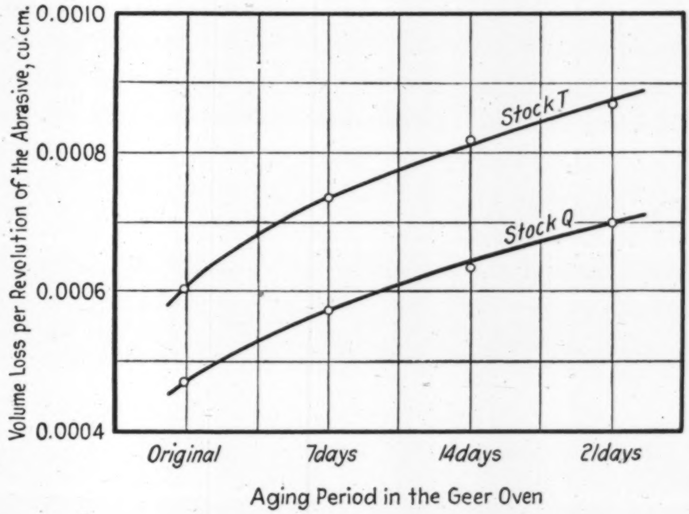

FIG. 9.—Showing Effect of Aging on Resistance to Abrasion.
$\frac{1}{2}$-in. rings; abrasive driven; speed, 1450 ft. per minute; load, 18 lb.

The curves in both figures show that the results on undercured stocks are erratic. The relative order of abrasion resistance is reversed in several cases. The fact that these laboratory tests on undercured stocks do not give a true picture of the relative resistance to wear on the road has been demonstrated by several road tests on tires having the properly cured tread on one-half and the undercured tread on the other half of the tire. The undercured tire showed more rapid wear. When testing undercured stocks on the machine, the abraded rubber becomes soft and gummy and tends to hang to the specimen in shreds, preventing further abrasion. This leads to poor checks and erroneous results. It has been shown by numerous tests that slightly overcured stocks give more uniform and comparable results than undercured stocks.

*Effect of Aging:*

Figure 9 shows the effect of Geer oven aging on two stocks. These stocks, Q and T, were shown above to be sensitive to overcure, and rather rapid aging would therefore be expected, especially as the test rings were given a slight overcure. The curves show a marked decrease in abrasion resistance as the stocks age. It has been found, however, that some natural aging after cure is desirable in that more uniform test results are obtained. The practice of aging all test specimens at least 48 hours after cure has been adopted.

*Results Obtained with Different Mixes of the Same Stocks:*

The following table shows the results obtained with four lots each of four compounds. One lot of each was factory mixed and three lots were mixed in the laboratory. The factory mixes were called lot No. 1. Lots Nos. 2 and 3 were laboratory mixes using black master batches and following the procedure recommended by the Physical Test Committee of the Rubber Division, American Chemical Society. Lot No. 4 was made in the laboratory using the same method of mixing but adding the black as a dry pigment. All of these tests were run on rings $\frac{3}{8}$ in. wide with the specimen driven. Each figure is the average of at least six check tests on different days, using rings cured in the mold at the same time. The indices calculated from the volume losses are given in parentheses.

| | VOLUME LOSSES, CU. CM. | | | |
|---|---|---|---|---|
| | COMPOUND M | COMPOUND N | COMPOUND O | COMPOUND P |
| Lot No 1—Factory mix... | 0.387 (100) | 0.409 (100) | 0.527 (100) | 0.304 (100) |
| Lot No. 2—Laboratory mix — black master batch... | 0.323 (120) | 0.356 (115) | 0.462 (114) | 0.253 (120) |
| Lot No. 3—Check on lot No. 2.................. | 0.343 (113) | 0.385 (106) | 0.429 (123) | 0.214 (142) |
| Lot No. 4—Direct black mix in laboratory....... | 0.456 (85) | 0.508 (80.5) | 0.618 (85) | 0.378 (80.5) |

It is assumed that the laboratory mixes using the master batch give stock which represents the most thorough and uniform mixing of the group. The results show that these mixes gave rings which were more abrasion resisting than either of the factory mixes which also used master batched black or the laboratory mixes where the black was incorporated directly. Of the two latter, the factory mixes were superior in abrasion resistance to the direct mixed laboratory batches. It is evident that the improved dispersion of the black due to the use of master batches is reflected in the abrasion results. This indicates that thoroughness of mixing and degree of dispersion of the carbon black is of great significance in abrasion testing and emphasizes the necessity for uniform methods of processing stocks which are to be compared.

*Comparison of Laboratory Tests with Road Tests:*

The following table is a summary of some comparisons between laboratory and road tests. All stocks run in groups at the same time are designated by lot numbers. The several compounds in each group are given letters but these do not refer to the same compounds previously discussed, except as noted, and the same letters do not necessarily indicate similar compounds:

| | COMPOUND | INDEX | |
|---|---|---|---|
| | | LABORATORY | ROAD |
| Lot No. 1 | A (Same as compound L above) | 100.0 | 100.0 |
| | B (Same as compound K above) | 105.0 | 106.0 |
| | C (Same as compound J above) | 110.0 | 116.0 |
| | D (Same as compound H above) | 126.0 | 127.0 |
| Lot No. 2 | A | 124.5 | 121.0 |
| | B | 117.0 | 105.3 |
| | C | 100.0 | 100.0 |
| Lot No. 3 | A | 100.0 | 100.0 |
| | B | 133.0 | 139.7 |
| Lot No. 4 | A | 100.0 | 100.0 |
| | B | 110.0 | 102.8 |
| Lot No. 5 | A | 100.0 | 100.0 |
| | B | 114.4 | 108.0 |
| Lot No. 6 | A | 100.0 | 100.0 |
| | B | 115.5 | 108.5 |
| Lot No. 7 | A (Same as compound M above) | 100.0 | 100.0 |
| | B (Same as compound N above) | 81.7 | 84.1 |
| | C (Same as compound O above) | 65.7 | 65.6 |
| Lot No. 8 | A | 100.0 | 100.0 |
| | B | 109.0 | 109.5 |
| Lot No. 9 | A | 100.0 | 100.0 |
| | B | 100.5 | 93.0 |
| Lot No. 10 | A | 100.0 | 100.0 |
| | B | 119.6 | 106.4 |

| | COMPOUND | INDEX<br>LABORATORY | ROAD |
|---|---|---|---|
| Lot No. 11 | A | 100.0 | 100.0 |
| | B | 141.0 | 121.0 |
| Lot No. 12 | A | 100.0 | 100.0 |
| | B | 93.6 | 96.7 |
| Lot No. 13 | A | 100.0 | 100.0 |
| | B | 109.1 | 101.0 |
| Lot No. 14 | A | 100.0 | 100.0 |
| | B | 106.5 | 101.4 |
| Lot No. 15 | A | 100.0 | 100.0 |
| | B | 89.4 | 90.5 |

## CONCLUSIONS

1. This machine has given results on tread stocks comparable to the results obtained in road tests. Precautions were taken regarding mixing of experimental batches, state of cure, specimen width, test procedure and age of specimens.

2. It has been shown that this machine did not truly evaluate service wear of undercured stocks.

3. Abrasive wear increased with extended overcures and aging.

4. Abrasive wear resistance is influenced by method of mixing the compound, and the degree of dispersion of the reinforcing fillers.

5. This machine as set up is useful for development purposes and may be used to replace service testing in some of the preliminary work now done on the road.

*Acknowledgment:*

The authors wish to acknowledge gratefully their indebtedness to C. W. Bedford and his associates for their helpful cooperation and particularly for the road test data which was furnished, to W. N. Jones for his suggestions and support which has made this work possible, and also to the personnel of the Goodrich Testing Laboratories for assistance in preparation of samples and equipment.

# THE MEASUREMENT OF THE WORK DONE IN PUNCHING A RUBBER CYLINDER FROM A TEST SHEET

BY HARLAN A. DEPEW,[1] S. I. HAMMOND[2] AND E. G. SNYDER[3]

## SYNOPSIS

An apparatus consisting of a pin working through a circular opening in a die has been constructed that measures the work done in punching at high speed a rubber cylinder from a test sheet. The test has been found to be essentially cutting. Shear, which requires rigidity, is not involved. The deformability of rubber causes the work of cutting to be large and an important factor in abrasion resistance.

A reasonably true cylinder can be punched from a sheet of rubber by a pin working through a steel die with a cylindrical opening. The mechanics of punching a cylinder seemed to be sufficiently like the tearing of a small piece of rubber from a tire tread that it was decided to measure the work done in the hope of obtaining an absolute method for determining abrasion resistance in the place of the relative methods that are in general use.

The first criterion in designing an apparatus for this purpose is that the force must be transmitted at high speed but under measurable conditions. Accordingly, the apparatus shown in Fig. 1 was built, consisting essentially of a pin and die, *B* and *C*, the pin being actuated by hydraulic pressure, which is produced by the movement of the piston *H* when the weight is released. By varying the size of the piston, weight, and lever arm, the rapidity with which the pressure in the system builds up can be changed. Two Crosby indicators are connected to the pressure line, the spring of the one at the right being removed and the shaft being connected directly to the punching pin so that the movement of the lever *S* is proportional to the pin travel, while the lever *R* on the other indicator simultaneously measures the hydraulic pressure on the system. The records of pressure or force and pin travel are recorded on a chart which is driven independently by a synchronous motor at a definite speed. The two records obtained are shown in Fig. 2. From these two curves, the factor of time is eliminated and a work diagram is drawn as shown in Fig. 3; the work being measured by the area between the curve and the pin travel axis.

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[3] Investigator, Rubber Section, Research Division, The New Jersey Zinc Co., Palmerton, Pa.

In operating the apparatus, a hydraulic head of a few inches of oil is placed on the apparatus by the pump *G* as shown by the oil level *L* in the gage. Considerable care is necessary to keep trapped air and grit from the hydraulic fluid which consisted of a 90–10 mix of light engine oil and kerosine.

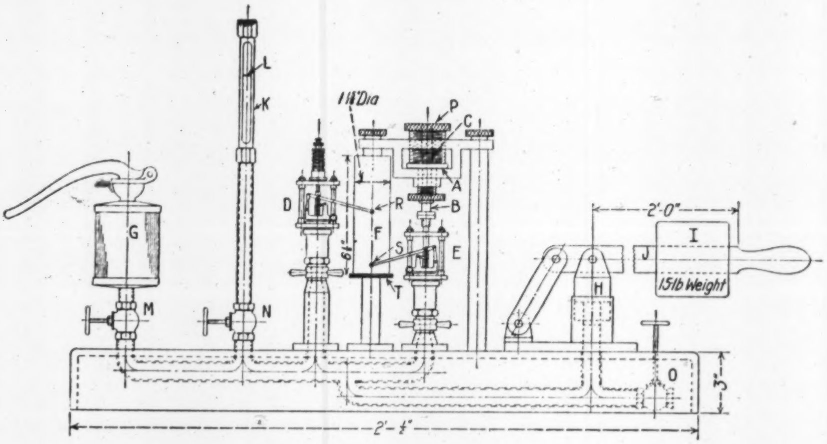

FIG. 1.—Apparatus for Recording Work of Punching Rubber Sheets.

*A*, Rubber test slab.
*B*, Pin (¼ in. diameter).
*C*, Die (¼ in. opening).
*D*, Standard Crosby indicator (style BC-101 Crosby steam gage and valve company).
*E*, Standard Crosby indicator with the spring removed and with the piston rod operating against the pin *B*.
*F*, Cylinder carrying chart (6 in. circumference, 4 r.p.s.).
*G*, Oil reservoir and hand pump.
*H*, Hydraulic piston.
*I*, Operating weight.
*J*, Lever transmitting force to hydraulic piston *H*.
*K*, Oil gage, glass front.
*L*, Oil level, giving constant oil pressure in the system.
*M*, Valve, shut off from reservoir.
*N*, Valve, shut off for gage.
*O*, Valve, outlet for clean-out, etc.
*P*, Tightening nut for holding specimen.
*R*, Pencil lever recording pressure.
*S*, Pencil lever recording the pin travel.
*T*, Gear driven by a synchronous motor.

## TEST RESULTS

In order to obtain some data on different types of compounds, five typical compounds were prepared (Table I) and comparisons were made using different states of cure and different thicknesses of test sheets. Figure 4 shows the effect of slab thickness on the work done and Fig. 5 shows the results obtained at two different thicknesses above the critical thickness. A very thin sheet stretches and may wedge between the pin and die so that the work done will be greater than for a thicker piece. This part of the curve is of no significance and accordingly slabs of at least 0.04 in. thickness must be used.

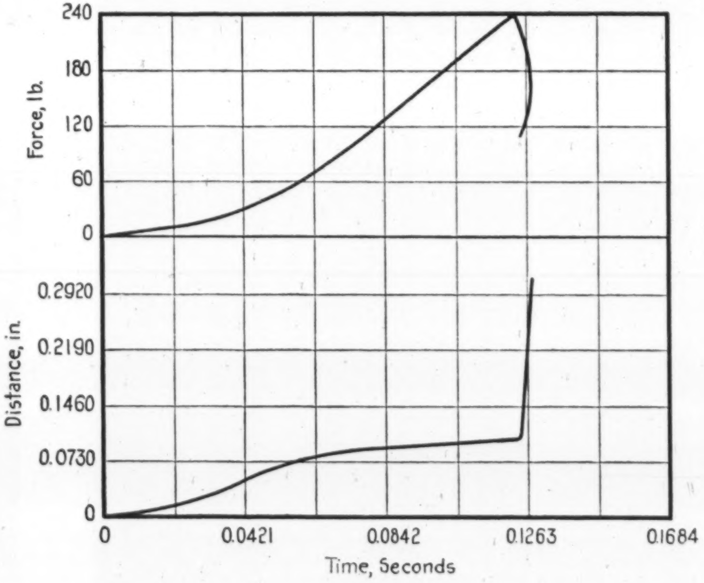

FIG. 2.—Showing Force-Time and Distance-Time Curves.

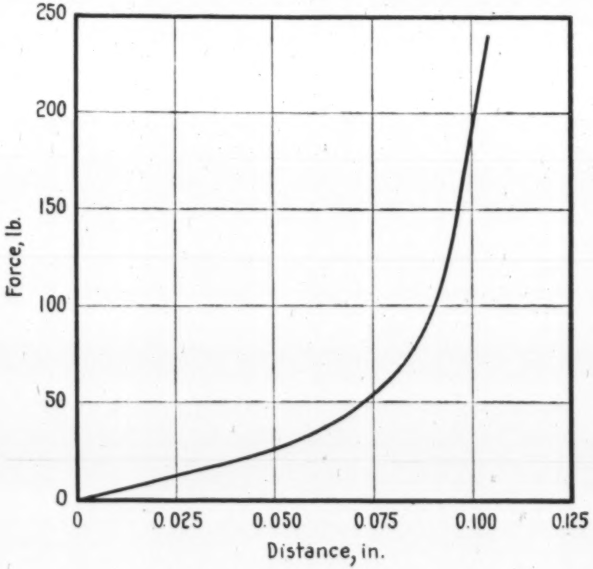

FIG. 3.—Showing Force-Distance Curve Obtained from Fig. 2.

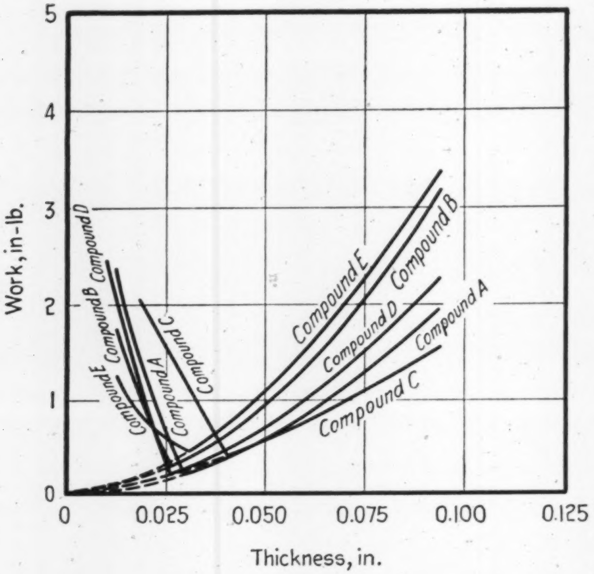

FIG. 4.—Showing Work Done at Optimum Cure in Punching Various Thicknesses of Sheets Listed in Table I.

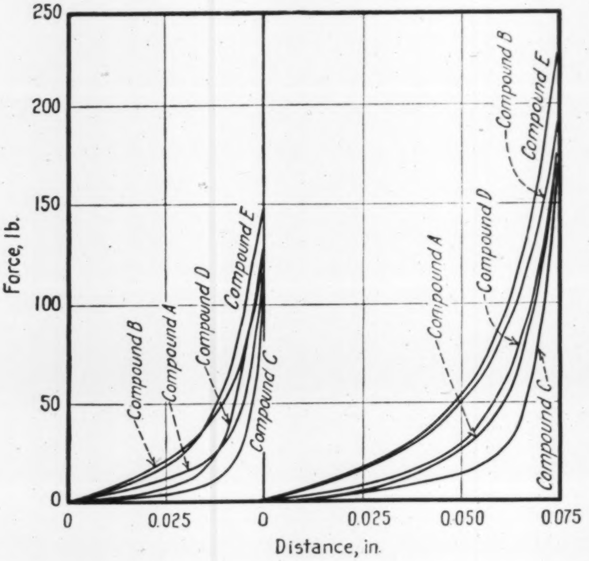

FIG. 5.—Showing Force-Distance Curves at Optimum Cure for Two Thicknesses.

The results, in a general way, correlate the work done with the hardness of the stock as shown in Fig. 6. Accordingly, it was to be expected that increasing cure would offer increasing resistance which is demonstrated in Fig. 7.

From these data it is apparent that the test is essentially a cutting test, rather than a combined cutting and tearing test and cannot be used for the direct measurement of abrasion resistance, although it is helpful in understanding abrasion. The results are probably similar to those of the Hippensteel-Bell Telephone Laboratories test except that the punching is done at relatively high speed.

In discussing abrasion resistance, the term "shear" is frequently brought forward as a factor and it is important not to confuse shear with this punching test. In pure shear, the stresses are concentrated in one plane and it is obvious that as soon as deformation takes place there is no longer pure shear but a combination of shear, tension and compression. In the case of a relatively non-deformable material, such as steel, the deformation is small and the shearing stresses therefore predominate over the secondary stresses, whereas in the case of a very deformable material, such as rubber (see Fig. 8), the secondary stresses predominate and the pure shearing stress represents only a small portion of the total.

TABLE I.—COMPOUNDS TESTED.

| Compound | Composition, parts by weight | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Pale Crepe | Smoked Sheet | XX Red Zinc Oxide | Red Label Kadox | Carbon Black | Diorthotolyl Guanidine | Stearic Acid | Medium Pine Tar Oil | Anti-oxidant | Whole Tire Reclaim | Sulfur | Mineral Rubber |
| A—Carbon Black Tread Compound | 50 | 50 | 18.2 | .. | 24.4 | 1.25 | 2 | 2 | .... | .. | 3.5 | ... |
| B—Reclaim Tread Compound .... | 30 | 30 | 18.2 | .. | 35 | 1.00 | 2 | 2 | .... | 66 | 3.5 | ... |
| C—Carcass Friction Compound ... | 46 | 46 | .... | 28 | .... | 0.90 | 0.45 | 1.80 | 1.80 | .. | 3.7 | 4.5 |
| D—Kadox Tread Compound....... | 46 | 46 | 56 | 56 | 6.9 | 0.35 | 0.90 | 0.90 | 0.90 | .. | 3.2 | ... |
| E—Solid Tire Compound.......... | 46 | 46 | 84 | 84 | 8.7 | 0.60 | 0.23 | 0.46 | .... | .. | 6.4 | ... |

Since it is impossible to have pure shear when material deforms, it is impossible to have work expended in shearing but only an applied force. It is for the above reason that the work done on the rubber is referred to as the work of punching rather than the work of shearing.

Rubber has about the same compressibility as water. It only flows under load and does not stretch. In order to measure the approximate shearing resistance of rubber, it would be necessary to confine the rubber so as to reduce this flow to a very small amount. This might be accomplished by vulcanizing the test specimen in a

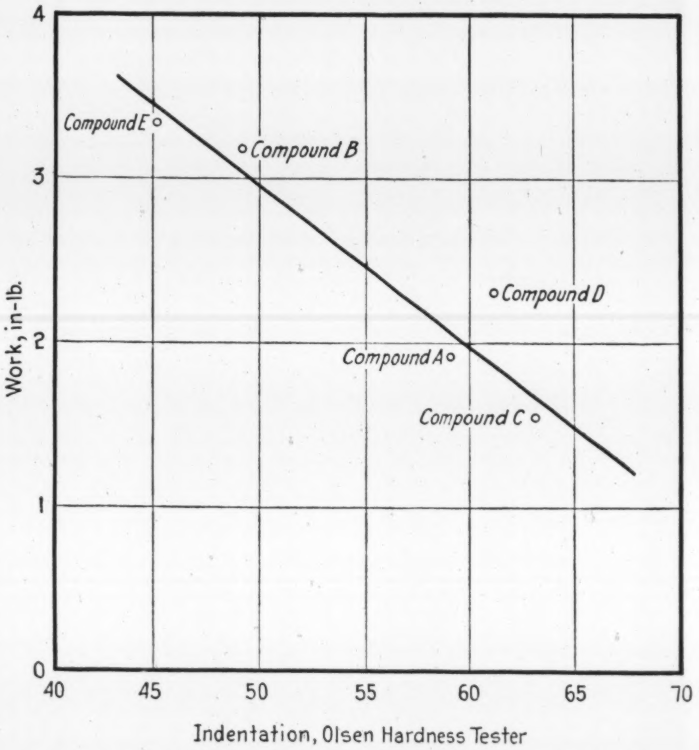

FIG. 6.—Showing Relation Between Work Done in Punching and the Hardness.

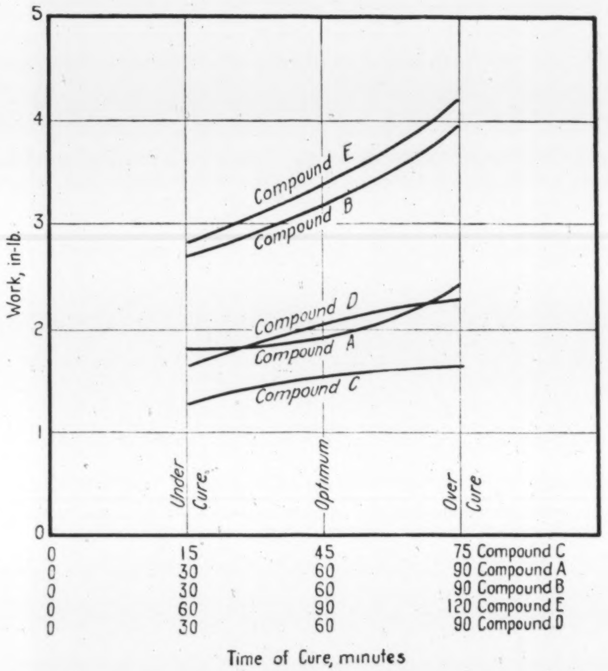

FIG. 7.—Showing Relation Between Work of Punching and Time of Cure.

a rigid metal shell to which it is tightly bonded, or in a hard rubber case which encloses it on all sides; cutting the shell or case at the shearing plane and then placing the test specimen under a shearing force at that plane.

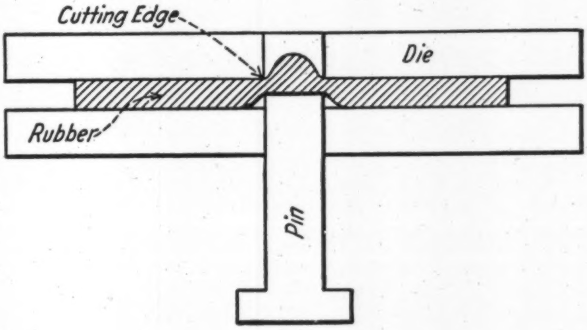

FIG. 8.—Diagrammatic Sketch Showing Movement of Rubber in the Die.

## CONCLUSION

In conclusion, the action of a pin in punching a rubber cylinder from a test sheet is essentially a cutting one. Since the rubber flows during cutting, the action is not shear but rather work of cutting. It is the flow of the rubber that makes the work of cutting large enough to make it an important factor in developing resistance to abrasion.

# ABRASION TESTING OF RUBBER WITH BUREAU OF STANDARDS TYPE MACHINE

By Warren E. Glancy[1]

## Synopsis

There are several types of machines used for the abrasion resistance of rubber which gives satisfactory results. Most of these, however, require comparatively large samples and were perhaps designed more for tire tread work than for general use.

This paper describes experience with a machine similar to that developed at the U. S. Bureau of Standards. The results obtained warrant considerable confidence in this machine. Figures are given which show good agreement between optimum cure as determined by tensile strength tests and by this abrasion method.

Road tests and machine tests show good agreement.

It is concluded that the advantages of this machine (and of the present Bureau of Standards machine) are such as to warrant greater interest on the part of rubber laboratories.

## Introduction

Sigler and Holt[2] have described the operation of an abrasion machine which was designed at the U. S. Bureau of Standards. Their experience with this machine has been so successful that they believe it gives a reliable indication of the wear of tire treads in service. In view of the lack of agreement on methods of abrasion testing, it has been thought worth while to offer comments on the use of a machine which is fundamentally the same as that of the Bureau and based upon a simple early model once used in the Bureau's rubber laboratory.

In getting this machine into operation some changes seemed to be necessary which were adopted. The essential changes were: (1) to provide a suitable means of removing grindings; and (2) to make the holding arm adjustable so as to permit the testing of specimens which might be of different thicknesses. With these changes, results were obtained which seem satisfactory.

The author's experience with abrasion machines had not been wholly successful. For this reason the present design, which seemed to offer some real advantages, was developed.

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[1] Manager of Laboratories, Hood Rubber Co., Inc., Watertown, Mass.

[2] P. A. Sigler and W. L. Holt, "A Simple Abrasion Test Machine for Rubber," *India Rubber World*, Vol. 82, No. 5, p. 63 (1930).

## DESCRIPTION OF MACHINE

The advantages to be noted in the machine described herein are as follows:

1. Simplicity of construction.

2. Results were not affected by other specimens placed in close proximity (as was the case with some machines in use at that time).

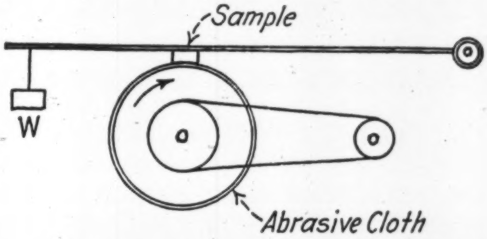

FIG. 1.—Diagramatic Sketch of Abrasion Machine.

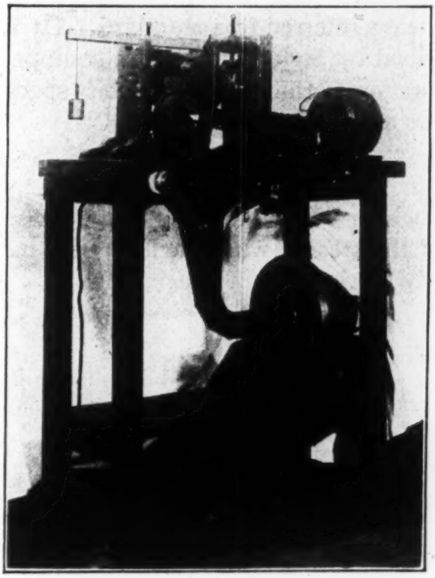

FIG. 2.—The Complete Machine.

3. The abrasive surface would not change due to the polishing or waxing effect of rubber specimens since a new surface was provided for each test.

4. Results could be obtained in five minutes or less.

5. As the test specimen was only an inch square, specimens could

be taken from non-skid tire designs or small articles such as molded heels.

6. The short time of operation, in addition to speeding the testing, also gave little time for producing excessive temperatures, or for coating the abrasive surface with waxy materials such as stearic acid.

Figures 1 and 2 show the principal features of the machine. Essentially it consists of a motor-driven pulley (such as used by cobblers for buffing shoes) which is covered with No. 2½ garnet abrasive cloth against which the test specimen is held with a force of 4 lb. The pulley is 6 in. in diameter and revolves at the rate of 30 r.p.m. A movable cylindrical brush is held against the abrasive surface and a strong suction concentrated at a point a little below the brush. The arm is made level before starting. The number of revolutions is counted by an ordinary tap counter. Usually the test is continued for 5 minutes although a shorter period is satisfactory on stocks which wear away as rapidly as heels.

Abrasion results are reported as loss in cubic centimeters per 1000 revolutions and always referred to a standard. (It will be noted that the machine described by Sigler and Holt is equipped with means for measuring the reduction in thickness of the test specimen and calculations of volume loss are eliminated).

## TEST RESULTS

A few values which bring out comparisons of service tests (road tests in the case of the treads) with machine tests may be of interest:

| | SERVICE INDEX | MACHINE INDEX |
|---|---|---|
| Tread No. 1 | 100 | 100[a] |
| Tread No. 2 | 160 | 149[a] |
| Black Heel No. 1 | 100 | 100 |
| Black Heel No. 2 | 95 | 91.8 |
| Tan Heel No. 1 | 95 | 87 |
| Tan Heel No. 2 | 100 | 100 |

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[a] In this case, the specimens used on the machine were taken from the worn tires, which were tested in California.

Due to the difficulty of getting a proper measure of service tests on heels, the above figures of service index are only approximate, but represent the conclusion of our test goods department. The conclusion was based on the average of 5 pairs of black and 4 pairs of tan heels, which were mismated.

Four sole stocks tested on this abrasion machine showed the following values:

| | MACHINE INDEX |
|---|---|
| Compound A | 100 |
| Compound B | 85.8 |
| Compound C | 75.8 |
| Compound D | 65.5 |

The stocks in question are rated in service in the same order and approximately in the same ratio to each other.

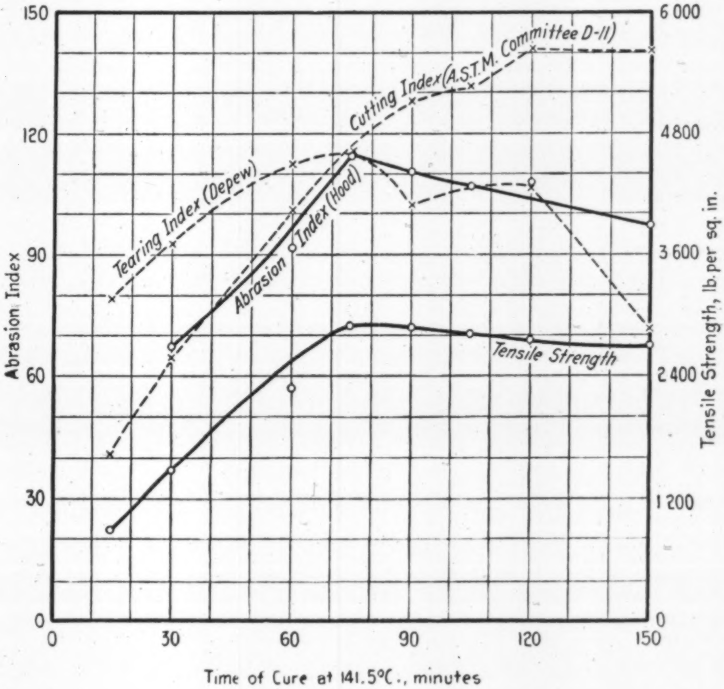

FIG. 3.—Relation Between Tensile Strength and Abrasion Index, and Time of Cure of High-Zinc-Oxide Stock.

At the suggestion of Mr. H. A. Depew, abrasion tests were conducted on the rubber mix previously used by him, reports of which have been published.[1] The recipe for this mix is as follows:

| | |
|---|---|
| Smoked sheet | 100 parts |
| Sulfur | 6 " |
| XX Red zinc oxide | 137 " |
| Hexa | 1 " |

Cures were made at 141.5° C. for different periods of time ranging up to 150 minutes. Abrasion tests in duplicate were run on each as well as tensile strength tests. The results are plotted and appear in Fig. 3, together with results obtained by Subcommittee XIV on Abrasion, of the Society's Committee D-11 on Rubber Products, which results are included for comparison. It is noticeable that the optimum cure as judged by tensile strength and abrasion on this machine is the same. Sigler and Holt obtain similar results.

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[1] H. A. Depew, "An Explanation of Some of the Difficulties in Abrasion Testing of Rubber," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 871 (1928).

For comparison with other data reported by Subcommittee XIV,[1] mixes of black tread stocks were made and vulcanized 60 minutes at 141.5° C. and then tested for abrasion resistance. The mixes are given in Table I. The abrasion index comparisons are given in Table II.

TABLE I.—MIXES OF TREAD STOCKS, PARTS BY WEIGHT.

| Compound | Pale Crepe | Smoked Sheet | Diortho-tolyl Guanidine | Sulfur | XX Red Zinc Oxide | Micronex | Stearic Acid D.P. | Medium Pine Tar | Whole Tire Reclaim |
|---|---|---|---|---|---|---|---|---|---|
| No. 5........... | 50 | 50 | 1.25 | 3.5 | 18.2 | 24.2 | 2 | 2 | .. |
| No. 6........... | 50 | 50 | 1.25 | 3.5 | 18.2 | 40.0 | 2 | 2 | . |
| No. 8........... | 40 | 40 | 1.12 | 3.5 | 18.2 | 37.5 | 2 | 2 | 33 |
| No. 9........... | 30 | 30 | 1.0 | 3.5 | 18.2 | 35 | 2 | 2 | 66 |
| No. 10.......... | 50 | 50 | 0.45 | 3.5 | 18.2 | 40 | 2 | 2 | .. |

TABLE II.—ABRASION INDEX COMPARISONS.

| Compound | Hood Laboratory Abrasion Resistance | Service Test Comparison Made by Subcommittee XIV | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | Summer Test No. 1 | Summer Test No. 2 | Summer Test No. 3 | Summer Test No. 4 | Summer Test No. 5 | Average of Five Summer Tests | Single Winter Test |
| No. 5............ | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 |
| No. 6............ | 91.6 | 103 | 89 | 85 | 98 | 102 | 95 | 99 |
| No. 8............ | 67 | 56 | 76 | 71 | 80 | 79 | 72 | 72 |
| No. 9............ | 62.8 | 52 | 41 | 52 | 71 | 60 | 55 | 62 |
| No. 10........... | 108.2 | 124 | 103 | 88 | 99 | 98 | 102 | 167 |

With the exception of stock No. 10, which is undercured and probably better wearing at winter temperatures, fair agreement is shown. The author concurs with the conclusions of Sigler and Holt that this type of machine is capable of producing very satisfactory results.

## COMMENTS

Temperatures of 100 to 110° F. may be expected at the surface of the test specimen during the run, when the room temperature is approximately 75° F. Such differences probably do not affect tests on properly cured stocks to any extent. The ability to control temperatures during the test might be a step in advance in evaluating stocks which are to be used under very hot or very cold conditions.

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[1] *India Rubber World*, Vol. 81, No. 2, p. 66, November 1, 1929.

In attaching the abrasive paper, a 1-in. strip is allowed to extend approximately one inch over the joint. This strip causes the arm to jump slightly and so produces an effect which simulates that of those machines which are equipped with lifts. Accordingly an abrasive action is obtained which resembles to some extent the tearing action described by Depew. Judging from the results noted, with undercures a cutting action is obtained and with overcures a tearing action. This might be expected in service.

Although a strong rubbing action is obtained by the cleaning brush, and although the suction is strong, an appreciable quantity of abraded particles sticks to the abrasive surface. Provided the quantity is not too great this may be an advantage in simulating road tests as there is always an appreciable amount of dust between the rubber and the ground.

One point of interest in the study of abrasive wear of rubber goods, which has perhaps not been emphasized as much as it might be, is the frictional resistance of the two surfaces in contact with each other. For instance, a high-grade sole may not last as long under some circumstances as one with less frictional resistance. More work would be done on a stock which brings a basketball player to a quick stop than on one which permits him to slide a little before stopping. The poorer stock would seem to give better service, but at the sacrifice of quick stopping.

# COMPARATIVE TESTS OF FOUR ABRASION MACHINES

## By C. A. Klaman[1]

## Synopsis

Tests using four different abrasion machines agreed with road service satisfactorily in the case of stocks cured to the optimum, but they did not correlate well in the case of undercured or overcured stocks. Accordingly, none of the machines can be considered suitable for specification work, but they are all desirable laboratory tools for studying rubber products where the state of cure can be determined.

The reproducibility of results was satisfactory, check results showing an average deviation from the accepted average of 3.5 per cent. The difference in reproducibility of results with different machines may have been in part due to more experience with some machines than with others.

The choice of an abrasion test method depends in part on the use to which the acquired information is to be put. If it is to be used in specification work, correlation with service is the first requisite, but if it is to be used as a tool for investigation, other considerations may become relatively more important.

Experience with four abrasion machines: du Pont,[2] Kelly,[3] United States Rubber Co.,[4] and New Jersey Zinc Co.[5] (Figs. 1 to 4) is presented to help answer questions in regard to correlation with service and reproducibility of results.

## Correlation with Service

Tests for correlation with service were confined to tire tread abrasive loss. Both laboratory results and service results on five compounds, together with formulas, are given in Table I, and show good correlation with compounds Nos. 5, 6, 8, and 9. The machines differ widely for the undercured stock (compound No. 10), and since cure may affect results so greatly, tests were made in a 20-volume zinc oxide stock, over a range of cures, as listed in Table II. A service test (Table I) has shown that an undercure does not lower tread life in one particular case, and unpublished data from this laboratory indicates that undercures in general develop nearly as high abrasion resistance as stocks cured to the optimum. It is generally agreed that overcured treads abrade badly in service, and the general effect of cure on abrasive service is shown in Fig. 5. The laboratory results from Table II are also shown in Fig. 5, and show lack of correlation

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[1] Research Investigator, The New Jersey Zinc Co., Palmerton, Pa.

[2] Ira Williams, *Industrial and Engineering Chemistry*, Vol. 19, p. 764, June, 1927.

[3] A. F. Hardman, W. L. MacKinnon and S. M. Jones, *New York Rubber Age*, Vol. 28, p. 463, February 10, 1931.

[4] R. T. Vanderbilt Co. Handbook, p. 72.

[5] H A. Depew, "An Explanation of Some of the Difficulties in Abrasion Testing of Rubber," *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part II, p. 871 (1928).

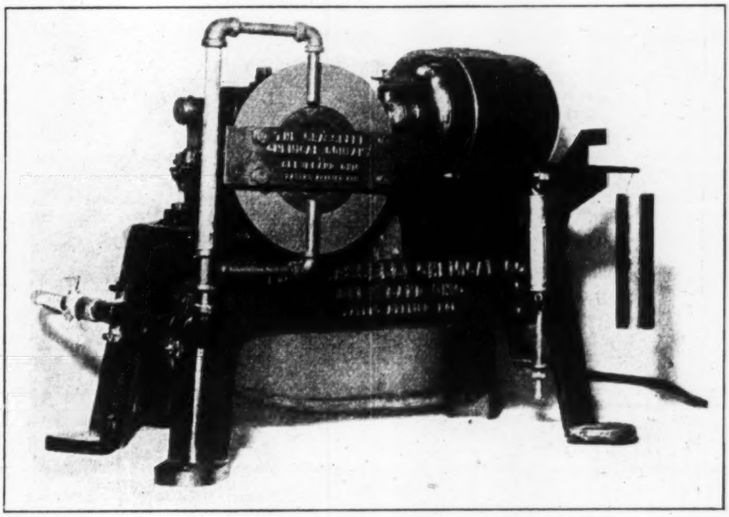

FIG. 1.—View of du Pont Abrasion Machine.

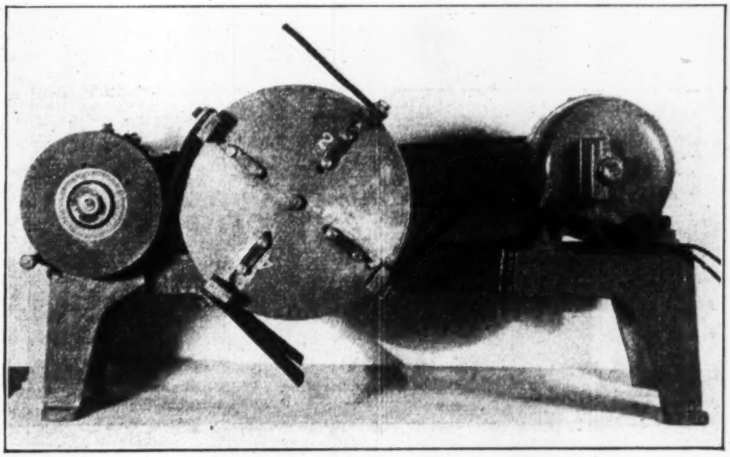

FIG. 2.—View of Kelly Abrasion Machine.

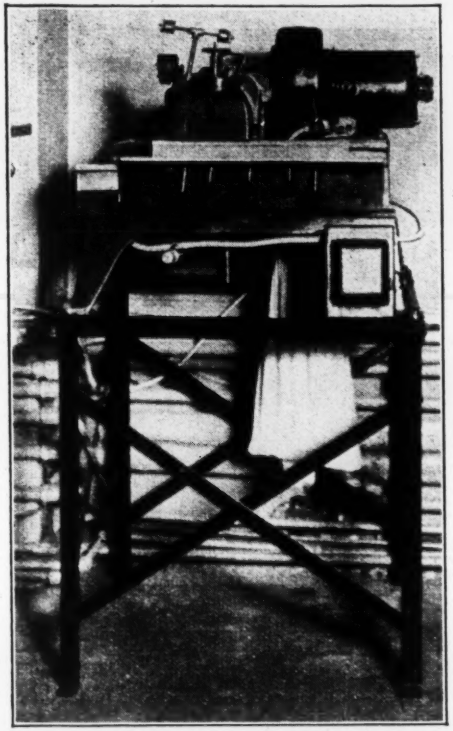

FIG. 3.—View of The United States Rubber Co. Abrasion Machine.

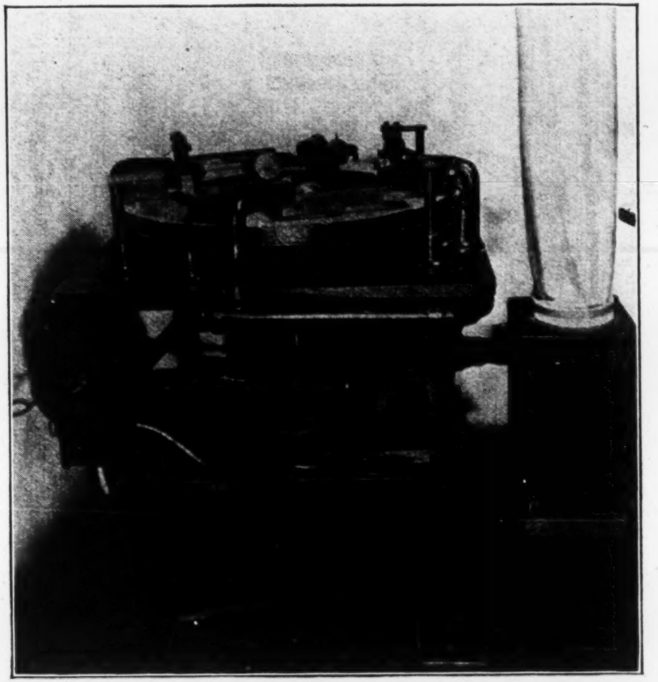

FIG. 4.—View of The New Jersey Zinc Co. Abrasion Machine.

TABLE I.—FORMULAS OF TREAD COMPOUNDS AND DIRECT COMPARISON OF LABORATORY TESTS WITH SERVICE TESTS.

| Compounds, Cured 60 minutes at 40 lb. | Composition, Parts by Weight | | | | | | | | | Service Tests[a] | du Pont Machine | United States Rubber Co. Machine | Kelly Machine | New Jersey Zinc Co. Machine |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Pale Crêpê | Smoked Sheet | Diorthotolyl Guanidine | Sulfur | XX Red Zinc Oxide | Micronex | Stearic Acid | Medium Pine Tar Oil | Whole Tire Reclaim | | | | | |
| No. 5—Low Abrasion Standard | 50 | 50 | 1.25 | 3.5 | 18.2 | 24.4 | 2 | 2 | .. | 100 | 100 | 100 | 100 | 100 |
| No. 6—High Abrasion Standard | 50 | 50 | 1.25 | 3.5 | 18.2 | 40 | 2 | 2 | .. | 102 | 92 | 103 | 108 | 116 |
| No. 8—Second Grade Tread.... | 40 | 40 | 1.12 | 3.5 | 18.2 | 37.5 | 2 | 2 | 33 | 79 | 78 | 81 | 84 | 84 |
| No. 9—Third Grade Tread..... | 30 | 30 | 1. | 3.5 | 18.2 | 35 | 2 | 2 | 66 | 60 | 68 | 74 | 77 | 66 |
| No. 10—Undercured Tread..... | 50 | 50 | 0.45 | 3.5 | 18.2 | 40 | 2 | 2 | .. | 98 | 120 | 79 | 59 | 51 |

[a] Report of Subcommittee XIV on Abrasion of Committee D-11 on Rubber Products. *India Rubcer World*, November 1, 1929, Vol. 81, No. 2, p. 61.

TABLE II.—TENSILE PROPERTIES OF A RUBBER COMPOUND OVER A RANGE OF CURES AND THE ABRASION RESISTANCE AS DETERMINED ON FOUR DIFFERENT MACHINES.

COMPOUND: 20 volumes of XX red zinc oxide to 100 volumes of rubber, 6 per cent of sulfur and 1 per cent Hexa by weight calculated on the rubber content.

| Time of Cure at 141.5° C., minutes | Tensile Strength, lb. per sq. in. | Elongation, per cent | Stress, lb. per sq. in. for Elongation of | | Abrasion Resistance | | | |
|---|---|---|---|---|---|---|---|---|
| | | | 300 per cent | 450 per cent | New Jersey Zinc Co. Machine | United States Rubber Co. Machine | Kelly Machine | du Pont Machin |
| 15 | 1440 | 625 | 335 | 705 | 66 | 45 | 59 | [a] |
| 30 | 2820 | 705 | 520 | 1070 | 80 | 59 | 64 | 90 |
| 60 | 3260 | 635 | 735 | 1540 | 95 | 91 | 91 | 92 |
| 75 | 3400 | 635 | 780 | 1670 | 100 | 100 | 100 | 100 |
| 90 | 3305 | 615 | 910 | 1710 | 104 | 109 | 108 | 94 |
| 105 | 3300 | 615 | 915 | 1760 | 95 | 111 | 111 | 93 |
| 120 | 3080 | 585 | 955 | 1870 | 93 | 114 | 98 | 89 |
| 150 | 2920 | 575 | 1050 | 1870 | 76 | 109 | 90 | 85 |
| 180 | 2280 | 505 | 1140 | 2020 | 49 | 106 | 80 | 80 |

[a] Pulled out of holder, probably because of softness.

TABLE III.—PERCENTAGE DEVIATION FROM ACCEPTED STANDARD.

| Machine | All Tests Made in One Day | | | | | One Test Daily for 10 Days | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Number of Tests | Maximum Deviation, per cent | | Average Deviation, per cent | Percentage of Samples Testing with Over 5 per cent Deviation | Number of Tests | Maximum Deviation, per cent | | Average Deviation, per cent | Percentage of Samples Testing with Over 5 per cent Deviation |
| | | Plus | Minus | | | | Plus | Minus | | |
| du Pont.................. | 18 | 12 | 15 | 8.0 | 83.3 | 10 | 6 | 7 | 2.8 | 30.0 |
| Kelly.................... | 87 | 7 | 5 | 2.5 | 4.6 | 30 | 11 | 5 | 4.1 | 30.0 |
| United States Rubber Co. | 28 | 6 | 5 | 2.5 | 3.6 | 40 | 8 | 9 | 3.4 | 15.0 |
| New Jersey Zinc Co...... | 36 | 6 | 7 | 2.4 | 8.3 | 30 | 6 | 3 | 1.8 | 3.3 |

on both undercured and overcured stocks among themselves and with the estimated service. Investigation of the lack of correlation with service on undercures and overcures on the New Jersey Zinc Co. machine shows that the condition of the track or abrasive surface influences results. A new track with sharp abrasive causes less wear on undercured stocks and rapid wear on overcures. (See Fig. 6.)

It follows that none of the laboratory abrasion machines are sufficiently reliable to be used alone in specification requirements. It is reasonable to believe, however, that their use may be worked out in conjunction with other tests that will specify comparable cures. In investigative work, the state of cure can be determined, and accordingly there can be no doubt that all the machines can become valuable tools.

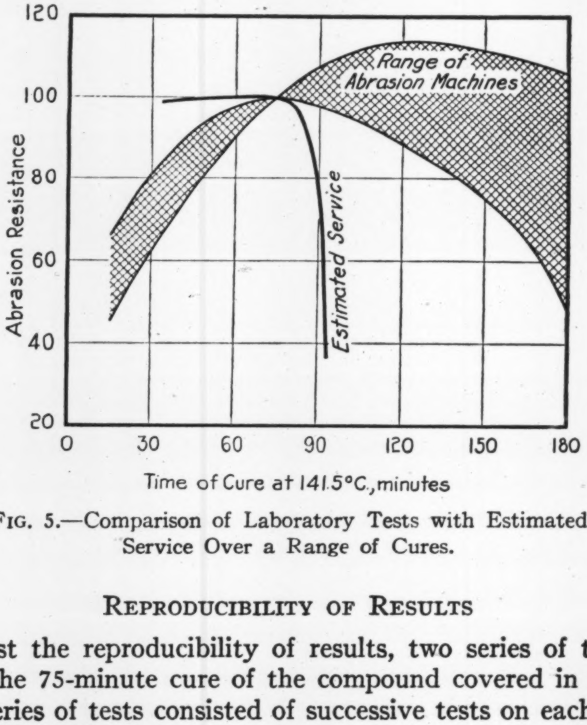

FIG. 5.—Comparison of Laboratory Tests with Estimated Service Over a Range of Cures.

## REPRODUCIBILITY OF RESULTS

To test the reproducibility of results, two series of tests were made on the 75-minute cure of the compound covered in Table II. The one series of tests consisted of successive tests on each machine for a full day. The other series of tests consisted of a single daily test on each machine for a period of ten days. In Table III are given the relative results.

The ability to reproduce results with any abrasion machine depends in part on the familiarity of the operator with the machine, and it should be noted that the operators have had more experience with the New Jersey Zinc Co. machine than with any of the others. By the same considerations, it is possible that the relative positions of some of the other machines might be bettered in another laboratory.

It seems reasonable that an investigator will obtain good results with any of these four machines and also with other machines such as the U. S. Bureau of Standards[1] machine and the angle[2] or slip[3] machines, provided that they are chosen with the limitations as to service correlation.

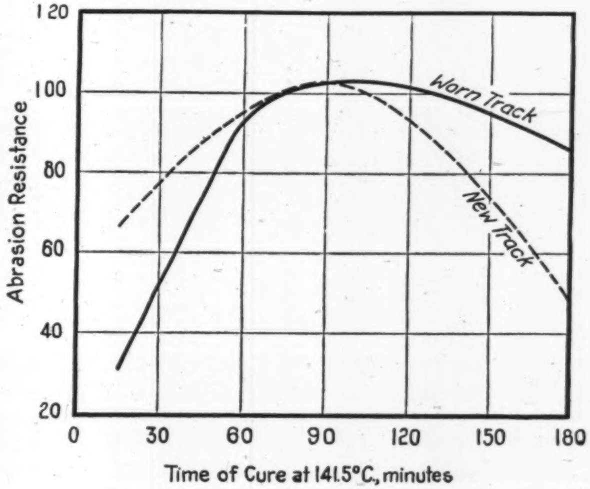

FIG. 6.—Abrasion Resistances Over a Range of Cures Comparing a New and an Old Crack.

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[1] P. A. Sigler and W. L. Holt, *India Rubber World*, Vol. 82, p. 63, August, 1930.

[2] W. W. Vogt, *Industrial and Engineering Chemistry*, Vol. 20, p. 302, March, 1928.

[3] L. J. Lambourn, *Transactions*, Inst. Rubber Industry, Vol. 4, p. 210 (1928).

# DISCUSSION ON ABRASION TESTING OF RUBBER

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MR. R. H. GERKE[1] (*presented in written form*).—Messrs. Carpenter and Sargisson have presented in their paper a comprehensive study of a specific tear test. Undoubtedly, they are aware of the common use of other methods and test specimens. I should like to present some data which show fundamental differences in different methods of measuring resistance to tear.

The pounds pull to rupture a rubber test specimen 1 in. in width was 6.8, 2.8 and 2.0 lb. for ¼, ½ and ¾ in. depth of single cuts, respectively, which were perpendicular to the direction of elongation. This evidence supports the conclusion of the authors that tear resistance appears to be correlated to the tensile strength. On the other hand, the resistance to tear of the same rubber with the same size test specimen was 1.2, 1.4, 2.0 and 1.6 lb., respectively, when the sample was slit lengthwise down the center, when the length of rubber resisting the tear was ¼, ½, 1 and 1½ in. In testing, the sample was opened up and the ends of the test specimen, ½ in. in width, were placed in the jaws of the testing machine. The tearing action took place halfway between the jaws. It is to be noted that the variations in results were systematic in the first test and not systematic in the second test.

THE CHAIRMAN (*Mr. H. A. Depew*[2]).—I believe that Mr. Gerke's remarks are in line with the experience of all of us.

Practically no group as large as this can get together without someone in the group taking the viewpoint that none of the machine tests is satisfactory and that hand tear is the only test.

MR. A. W. CARPENTER.[3]—Mr. Gerke made no comment as to the rate of application of force. In the Goodrich tear test we used the standard speed of jaw separation, 20 in. per minute. We have done some work on another type of test at a much higher speed, 60 in. per minute, and we find that the speed of pull is a factor which influences the numerical results.

MR. DEPEW.—I understand that Mr. Gerke used the standard tension testing machine. He took a strip of rubber 1 in. wide, cut it part way at right angles and then pulled it in the testing machine and the force to tear depended on the uncut width. When he took the strip and started tearing it lengthwise and gripped the two cut ends in the jaws of the testing machine, the force of tear was independent of the length to be torn.

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[1] General Laboratories, U. S. Rubber Co., Passaic, N. J.

[2] Chief, Rubber Section, Research Division, The New Jersey Zinc Co., Palmerton, Pa.

[3] Manager of Testing Laboratories, The B. F. Goodrich Co., Akron, Ohio.

MR. CARPENTER.—The latter is exactly the type of test we use with a speed of jaw separation of 60 in. per minute, and the speed makes a difference.

MR. F. H. AMON.[1]—I do not rise to defend the hand-tear method, although I do employ it.

Interesting results should be obtained if the elongation were determined as well as the load during tear of a number of stocks of different elongation and different modulus. Such work would be comparable to the work on the determination of the tensile strength calculated on the cross-sectional area at break. The thickness of the sheet at the tearing edge must differ considerably in zinc oxide stocks, carbon black stocks, and pure gum stocks. It would seem logical to calculate the load to tear for equivalent length of tearing edge.

In the hand-tear test I believe that we take the elongation into consideration, probably more or less unconsciously.

MR. DEPEW.—As a new method of attacking the problem of measuring tear resistance in our laboratory, a thin strip of metallic zinc to which rubber ordinarily adheres with difficulty was drilled with holes $\frac{1}{4}$ in. in diameter and spaced $\frac{1}{4}$ in. apart with smaller holes in between. A sheet of uncured rubber was placed on each side of the metal strip and vulcanized. The rubber flowed through the holes uniting the two strips of rubber with rubber buttons $\frac{1}{4}$ in. in diameter and with smaller buttons in between. The two ends of the strip were placed in the jaws of a tension testing machine and separated. Pieces of fabric vulcanized to the outer rubber surfaces prevented the rubber from stretching and the rubber buttons tore apart one after another with the small buttons absorbing the inertia energy of the pendulum of the testing machine. By this test method, one can average a large number of tears. Strangely enough we did not get as much reduction in tear with overcure as we had expected, except at high room temperatures. I should like to inquire if anyone has used test methods of this type in trying to evaluate tear?

[No further comments on the tear test were offered.—ED.]

MR. ALFRED HERZ.[2]—The abrasion test results from the testing man's point of view are rather erratic. Things do not seem to hang together, and I just wondered if the main cause of this discrepancy is not the heating of the rubber? That is, some of these rubber compounds apparently are better heat insulators than others, and you are probably heating, that is, softening the compound in one machine more than in another. It appears to me if there were some means provided for keeping the samples at a constant temperature while the abrasion is going on, one would probably get more consistent results; also, I wonder how much effect the particular abrasive material had on the results obtained. Certainly I take it for granted you always refer back to some known compound. Now, why is that necessary; why cannot the test be based on some material easily described or something of that sort that we can all go back to and therefor have results that are directed comparable.

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[1] Technical Director, Godfrey L. Cabot, Inc., Boston, Mass.

[2] Engineer of Tests, Public Service Company of Northern Illinois, Chicago, Ill.

MR. DEPEW.—All of our abrasion testing is based on a standard, and it is rather unfortunate that we should have to use a standard. The punching test described was an attempt to use quantitative units, and to get away from the so-called standard which is not a standard but a variable in every test.

MR. AMON.—I should like to ask if, in the punching test described by Messrs. Depew, Hammond and Snyder, the high modulus stocks cut more true cylinders than the soft stocks?

MR. DEPEW.—All stocks punch true cylinders at the speed at which that test was made (one-tenth of a second).

MR. HERZ.—I indicated my own idea on the variables—see if you cannot keep the point of abrasion at a constant temperature. I think it is the difference in temperature at the point of abrasion that causes the variation in the results. I should like to enter that as a criticism.

MR. DEPEW.—The simplest way to lower temperature would be to abrade at a lower rate, because if you abrade at high speed you are bound to generate considerable heat in a short time.

MR. HERZ.—How about water cooling of the wheels?

MR. DEPEW.—If you put your abrasive wheel actually in water, there is very little abrasive loss.

MR. HERZ.—I meant some internal circulation, a metal wheel with the abrasive mounted directly on it, and a circulation of cooling water through the wheel.

MR. DEPEW.—The United States Rubber Co. machine is that kind of machine; there is no water in it, but it is a metal wheel circulating with air passing it so that it is fairly well cooled by air cooling at that point.

MR. CARPENTER.—The Physical Testing Committee of the American Chemical Society reported considerable work with the U. S. Rubber Co. abrasion machine which was done at the U. S. Bureau of

Standards. The machine was put in a cabinet and kept at constant temperature. That is exactly what is referred to as cool at the point of abrasion. If Mr. Rupert would care to make any remarks, it might throw a little light on the situation.

MR. F. E. RUPERT.[1]—In the abrasion work carried on at the Bureau of Standards, we did find considerable difference in results due to differences in the temperature, but the differences due to humidity were not so great. I wish to ask Mr. Depew if he did any work at all on that punching machine at different temperatures? I am inclined to believe that you might get different results at higher temperatures.

MR. DEPEW.—The fact that the tests correlated so closely with hardness led us to believe that all punching tests would correlate with hardness, and that the change of hardness with temperature would be a guide as to the change in work of punching.

MR. RUPERT.—I think the question of temperature is very important, but it is very difficult to control. We were not able to control the temperature at the point of abrasion in any case. With the du Pont machine we controlled the air blast going across the abrasive but found that there was a rise in temperature of the test specimen itself. By taking temperatures at intervals during the course of the abrasion test it was found that the temperature of the test specimen rose rapidly at first and then remained constant during the remainder of the test. This difference in temperature between the test specimen and the constant temperature cabinet containing the apparatus appeared to be quite constant depending upon the temperature at which the test was made.

Only two machines were used during the investigation of the effect of humidity and temperature, the U. S. Rubber Co. machine and the du Pont machine. The tests were made by placing the instruments in a constant temperature cabinet.

MR. DEPEW.—The question of temperature is important, as is shown by the fact that it is recognized that winter wear of tires is perhaps five times as long as summer wear.

MR. D. C. SCOTT.[2]—I am interested in the question of rubber abrasion machines and have been for some years. I have in that time investigated this question of abrasives and have also tried file-cut steel wheels, using various speeds. I have tried solid abrasives, the ordinary grinding wheels of carborundum, emery, etc., both hard and soft grades. In connection with this I once talked to a certain professor of the Massachusetts Institute of Technology who had made an investigation of the various abrasive cloths and papers. There is nothing obtainable that is quite so even by microscopic measurement of abrasive size as the average run of commercial cloths, particularly in the carborundum and emery cloths. They run so remarkably uniform that we have since tried to work along the lines of a machine wherein the abrasive can be replaced each time with each set of samples, and I think the results that have been reported prove quite conclusively that there is a certain dependence that can be placed upon a machine with a replaceable abrasive of that sort. Under this condition, each sample starts off the same—perhaps theoretically not exactly the same, but we are talking of a practical laboratory test. Certainly a new sheet of emery paper used each test on a wheel such as is used on the U. S. Rubber Co. machine for instance, which is an aluminum wheel having a flanged web to dispense heat rapidly, makes for a good combination of conditions when combined with operation wherein the specimens held on the sliding table pass under the wheel and are given a definite period of rest from abrasion, keeping the rise in temperature negligible. The New Jersey Zinc Co. machine with the lift and drop attachment, to my mind, has a very good feature, in connection with this cooling of the specimen, and the Bureau of Standards machine, as described in Mr. Glancy's paper, certainly has merit, except that it keeps the sample in contact all the time and must generate considerable heat at the point of contact. I think, perhaps, by taking sugestions from all the machines we could find a satisfactory test by bringing in the good points of all of them. I am not by any means excepting the Grasselli machine, which we also manufacture. I think a machine could be designed around all these points, but like so many other things, the commercial angle of it discourages the manufacturer. I have had rather disastrous experiences in this connection with abrasion testing machines.

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[1] Chemist, Anaconda Wire and Cable Co., Pawtucket, R. I.

[2] Treasurer, Henry L. Scott Co., Providence, R. I.

There is one machine that has not been mentioned, that is the Sproull-Evans machine with the samples rotated under a static head of loose abrasive, and to my mind that has more good points than many others. Mr. Carpenter has, no doubt, had a lot of experience with that machine in his laboratory, and I am rather surprised that he has not mentioned it here. I should like to ask Mr. Carpenter if there is any reason why that has not been brought up.

MR. CARPENTER.—I do not know of any particular reason. The machine as designed by Mr. J. C. Sproull has been used a great deal and is still being used for certain types of stock and for certain kinds of services. I would not care to be too specific in commenting on this machine right now without some data in hand to support my statements, but for certain types of services the machine is quite satisfactory. Our experience, however, has been that the abrasion tests on this machine are decidedly influenced by the hardness factor of the stock. For instance, a pure gum rubber-sulfur compound may indicate much superior abrasive resistance on that machine to a high-grade tread stock. That is an extreme condition, but I think we all know that if we put a pure gum tread on an automobile tire, we would not get such superior results in service.

MR. SCOTT.—But is not that rather along the line of the experience with all abrasion machines that we have? Mr. Depew once sent me a sample of stock and also some very much under-cured stock at another time, which gave a very high abrasion resistance on the machine and it was an absolutely fictitious value—but the facts were there according to the machine readings as taken.

MR. CARPENTER.—We believe that the reason the under-cured stocks give high values is because they are gummy and induce slippage; that is, they have a lubricating effect due to the nature of the material. They lubricate the contact.

Mr. Glancy brought out a point in connection with contact, that is the relation between the material being abraded and the material doing the abrading, and intimated that this had not been studied as much as is desirable. We believe there is one way in which the factor of hardness can be introduced so as to give a condition paralleling that of service, namely, by using a thick enough specimen in the abrasion test, so that the stock itself will be free to bend and in that way introduce the element of modulus into the test. We know that a tire tread, for instance, is deformed on the road. It grips the road and is stressed. The rubber compound is actually stretched and bent; the tread moves and slides on the road and the degree of strain adjusts according to the modulus of the stock. We think that influence of modulus must also be allowed to enter into the laboratory tests if we are going to eliminate misleading effects from hardness of the cured compound; that is why we used a fairly thick specimen and an angle-type machine.

MR. F. D. ABBOTT.[1]—Mr. Depew and the Society's Committee D-11 on Rubber Products are to be congratulated on holding this Symposium on Abrasion Testing of Rubber, for a problem so vital to the tire manufacturer can find little hope for solution in widely scattered individual papers and discussions. Proof of this is found in the large number of excellent papers, each describing a machine of widely different design but with no general acceptance of any as the solution to the problem. We have heard a discussion of, or reference made to, at least eight different machines. Undoubtedly they cannot all satisfactorily evaluate tire treads. Without any attempt at criticism of the various machines, I shall take advantage of the plea that has been made for ideas concerning the design of an ideal abrasion machine. In the beginning we must grant that probably no laboratory device will satisfactorily evaluate the effect of various tread designs. However we can approach this by allowing proper deformation of the test sample as suggested by Mr. Carpenter.

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[1] Development Compounder, Mechanical Goods Division, The Goodyear Tire and Rubber Co., Akron, Ohio.

There are two prime requisites of the ideal abrasion machine, namely, rugged simplicity and accuracy both in duplicating results on test specimens taken from a given sample and also in duplicating service tests. This is true whether the machine be used for abrasion tests on tire treads or on rubber or miscellaneous materials undergoing a variety of service conditions. There are, however, a number of important factors involved in the design of a machine to duplicate service tests. This discussion necessarily pertains only to evaluation of tire treads.

There seem to be more advocates of an angle machine than of any other type. Although this type introduces a certain kind of slip, a study of abrasion of tires on the road indicates that non-angular slip is probably the greatest factor, particularly in the wear on rear tires. This will also shift to other tires in the new "front drives" such as the Cord car. Road wear is due entirely to overcoming the friction between the tire and the road surface. This friction is dependent on a number of external forces varying greatly in nature and amount. Thus road wear results primarily from:

1. Linear slippage due to fast traffic (quick starting and stopping resulting in slippage, mostly in the line of travel). Ordinary wear of properly aligned rear tires on straight runs also falls in this category of non-angular slip and wear.

2. Camber and toe-in of front wheels. (This wear is due to angularity.)

3. Wiping due to underinflation and other causes. Here deformation of the rubber becomes a big factor. (This deformation factor is important even in properly inflated tires.) (Both angular and linear slip are involved.)

4. Sidesway, going around curves, etc.

The ideal machine must "average these factors" by means of the principles embodied in its design, yet without any loss in rugged simplicity or accuracy in check tests. In other words the machine must simulate each of the above types of road wear, each to such an extent that the "average action" truly represents road wear. The papers presented have not described a machine incorporating these motions and the resulting wear.

The design of the Bureau of Standards machine described in the paper by W. E. Glancy would serve as a suitable starting point. If, in Fig. 1 of that paper, the abrasion disk were mounted off-center (thus giving constant linear slip and an intermittent deformation as in a car "settling" over a bump but not having left the road) and also if the shaft on which it is mounted were set at an angle to the drive shaft (thus giving angular slip similar to that resulting from camber, toe-in, or sidesway) one would have the fundamentals necessary to simulate all of the important types of action of a tire on the road. These factors are "averaged" by the proper choice of eccentricity and angularity. Such a machine, peculiarly novel in design, yet remarkable for its simplicity and accuracy, has been developed by the speaker (patents assigned to Firestone Tire and Rubber Co.) and tried over a number of months but I regret that permission is not yet granted to publish full details of design and tests.

There are a number of other factors to be considered even in such an ideal design:

1. The device must not be too sensitive to minor variations in temperature, size of test specimen (within ± a few thousandths of an inch), or hardness (stiffness) of the rubber tested.

2. It must be capable of running a standard simultaneously with the unknown (or check tests on the sample).

3. It must test samples of such size that these can be cut either from laboratory slabs or from the tread of new or worn tires. The samples must be of sufficient thickness above the edges of the holder to allow sufficient cushion and deformation as in service. Such deformations are simulated only by this new machine described.

4. The abraded material should be removed by gravity, or a brush on the abrasive, or preferably by suction.

5. The machine must be provided with abrasives of non-changing abrading ability. (It is possible to get stones to check standards after several thousand tests.) The abrading surface should be larger than that of the test specimen (as in service).

6. Proper choice must be made of the speed of rubbing and the type or grade of abrasive.

It is hoped that a more complete description together with data can be published in the near future.

In conclusion I have a comment on Mr. Depew's theory that by means of cutting, shearing and tearing tests we may be able to determine abrasion resistance of tread stocks. In some studies on cutting resistance I found that a certain high zinc oxide (Kadox) stock resisted cutting by a sharp tool under static loads to a much greater extent than any present-day tread stock. This stock was not particularly resistant to abrasion. Accordingly I feel that such tests will serve only to classify tread stocks into two or a maximum of three types. Further, even the "ideal" machine described should test all treads of different types according to conditions set up for that type.

MR. R. G. ROSHONG.[1]—In response to the question concerning improvements for the ideal abrading machine, it is my thought that considerable improvement could be effected by a continuous renewal of the abrading media. Our company is particularly interested in the abrasion resistance of two products, namely, rubber-covered electrical cords and soft tread caster wheels for vacuum cleaners. Concerning the first, we found there was considerable difficulty in keeping the abrading medium clean when using an abrasion machine consisting of two alundum wheels revolving in opposite directions at alternate periods over the same spot on rubber-covered cords. Apparently some of the rubber compounds contained substances which, upon abrasion, lubricated the alundum wheels, and this lubricating effect could not be entirely counteracted either by dissolving out with carbon tetrachloride or by brushing.

We now carry out an abrasion test by dragging the various cord samples behind a truck over a fifteen mile course of concrete road. This test seems rather crude, but by alternately spacing the accepted control samples and the cores to be tested, we are able to draw fairly good conclusions as to their relative abrasion-resisting properties.

Referring to the carrier wheels or caster wheels, we have designed a machine which is very similar in appearance to the one shown in Fig. 4 of Mr. Klaman's paper, constructed by the New Jersey Zinc Co. It consists of a circular table which revolves in a horizontal plane three-quarters of a revolution and then reverses. The wheel to be tested is held in a rigid standard off to one side of the table, 6 in. from the center, and toes in at an angle of 10 deg. to the tangent at point of contact. The speed of the revolving table is approximately $1\frac{1}{4}$ ft. per second and the abrading medium is a disk of 2/0 garnet paper. The garnet paper is removable and is renewed at the start of each test. It is felt that this particular machine does away with the heating effect to a certain degree, but not entirely. I say this because the sample is revolved and there is no continuous contact at any one point.

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[1] Chief Metallurgist, The Hoover Co., Canton, Ohio.

The results obtained on this apparatus have compared favorably with those obtained on field tests for caster wheels. It also checks relatively the results obtained by manufacturers of rubber products, who use the tumbling barrel test for abrasion resistance, where the pieces of rubber are tumbled in a barrel containing definitely graded sand or other abrading material.

MR. SCOTT.—There is another point we should take into consideration in this question of abrasion. The question of tire tread has been brought up here pretty strongly, but it seems as though there is a lot of rubber used in other ways than on the tread of tires to which this problem applies. It has been my experience in the work I have done, covering several machines, that the surface supporting the test sample has a decided influence and that it must play a very important part, particularly with treads of low-pressure tires, under the grabbing, sliding action we get against the road. The road surfaces, of course, vary; an overcured stock we all know will run indefinitely on a city street, but only a few miles on a sand road, and still every one of these machines mounts a piece of rubber, either cut from the surface of the material or from the laboratory slab, upon a solid backing. There is very little chance in any of the machines for any cushioning effect such as we get in service conditions on the tire, and which we do not get on the cover of a conveyor belt or the heel of a shoe or other things. I think that is one very big influence in the results you get in any of the machines.

MR. CARPENTER.—Along the line of Mr. Scott's remarks, we heartily concur in his statements. There is in road testing of tires a great variation in results dependent on the nature of the road surface. The sections of road to be used for road tests must be chosen carefully to attempt to keep conditions somewhat standard. That work is very well carried out by our tire design division and they get different results in Florida from those in the vicinity of Akron. Still different results are obtained in California. For that reason, in order to evaluate some of our treads, a national road test was instigated. It makes a lot of difference whether the road is wet or dry, and whether it is asphalt or concrete and what type of material is used in the concrete, whether it be a sharp sand or washed sand or any of the various other materials used in road building. It makes a difference whether the road is a loose gravel, crushed limestone or ordinary dirt road. Temperature enters in undoubtedly.

The big difficulty in evaluating stock in service has been the number of variables that are involved which are not under the control of the test engineer and which are not capable of thorough analysis. After all, we are concerned primarily with relative data. Expense of complete road tests under all conditions so as to evaluate thoroughly the compounds used in a tire line is justifiable once but not on every minor change that is required. Therefore, if we can do our development work on a comparative basis using a laboratory test and eliminate much of the road testing, there is a very attractive field for savings.

I want to say also in connection with Mr. Scott's remarks, that other rubber articles are as important as tire treads, but we are always obliged in studying a problem of that nature to consider the economic phases of it. In actual wear testing, the amount of money spent today is many times greater in the case of evaluation of tread wear than for any other single product. Most of the efforts which have been described do deal with treads, because it is economically the most attractive field. In remunerative value to the rubber manufacturer, the biggest results in dollars and cents saved will be shown if we can secure a satisfactory laboratory test which will represent service wear of treads and reduce the amount of actual road testing required.

I think Mr. Scott also is entirely correct in emphasizing the need of cushioning under stocks that are subjected to abrasion tests. That is an extension of the idea I mentioned before, of letting the modulus of the stock play a part. I may say now that Mr. Tronson and I hope, at a later date, to have a further communication to offer to this Society on abrasion testing in which that point will be emphasized.

MR. R. E. HUGHES.[1]—The du Pont abrader does not evaluate the quality in terms of service rendered by a heel stock. Our results show, using the du Pont abrader, a smaller loss per horse-power hour on a heel stock than it does on a first-grade tread stock. We know that the first-grade tread stock will wear longer as a heel than our heel stock.

An abrasion machine which will evaluate abrasive quality must respond to the resiliency of the stock in a way that will give comparable results to actual service.

MR. DEPEW.—The variance in abrasive wear with conditions of service is the reason that Subcommittee XIV of Committee D-11 has considered tear and cutting resistance to be so important that they were included in the symposium, in the hope of getting at the factors that make up abrasion and of evaluating them independently. We can heavily compound a stock so as to get high cutting resistance, but the tear resistance will be low, and we can get high tear resistance in a gum stock but the cutting resistance will be too low. Mr. Glancy's paper discussed tests on foot wear, and Mr. Hughes has told of his experience with heels and soles. Apparently heavy loading can influence abrasion tests through its effect on hardness.

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[1] International Shoe Co., Hannibal, Mo.

MR. SCOTT.—There is a point in all this that I want to bring up, and that is, we get certain results out of our machines from many of these tests that we know are wrong from our knowledge of the material, its ingredients, its cure, or something else, and any of these laboratory tests taken by themselves mean very little if we do not know the rest of the story. To complete the story that can be obtained in the laboratory, whether we get it by a tear test, cutting test, punch test, or a rubbing test, we need to know all the facts before we can accept any one result as indicative of the use to which the stock can be put.

MR. DEPEW.—In connection with our plans for this symposium, we discussed the possibility of getting a paper on the use of the microscope for indicating the abrasion resistance of the compound. Some technologists have indicated that the microscope can be of great help in evaluation.

MR. O. T. REES.[1]—One point in connection with these tests should receive some consideration. In the various methods of testing it appears that the results are concordant for the regular stock and the differences seem to develop between the different methods on the overcured and undercured stock. I am wondering if it would not be possible to carefully analyze these differences and develop something which could be worked into a machine that would test all types of stocks and give concordant results for all.

What I mean is this: Take the results from the different machines as outlined, which results are concordant for regular stocks, but for overcured stocks or undercured stocks they differ widely. There appears to be some feature in each test that makes one abnormally high or low. If the cause of these variations could be determined and equipment developed to correct for these differences, and this equipment incorporated in some testing machine, the results should then be concordant for all tests.

MR. DEPEW.—In other words, incorporate features in the abrasion test that will bring out the fundamental requirements of balances between the different factors that enter into abrasion testing.

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[1] Chief Chemist, Atchison, Topeka & Santa Fé Railway Co., Topeka, Kans.

MR. CARPENTER.—I think Mr. Scott brought that out also when he emphasized the need for knowing the whole story.

(*Closure by letter*).—We have heard in this symposium and the discussion which followed, descriptions of numerous laboratory machines which have been used for testing resistance of rubber compounds to abrasive wear as well as reference to several kinds of service tests. Various factors which influence the laboratory results have been considered in their relation as possible causes of discrepancies between these results and those in actual service. It is quite obvious from the remarks that none of the methods currently used is entirely satisfactory. The problem of laboratory evaluation of abrasive wear of rubber has not been solved. The purpose of this symposium has been realized, however, for attention has been focused on the requirements for an ideal abrasion test and numerous suggestions have been called forth. It is hoped that increased effort and inspiration in the study of the problem will result to the end that greater improvement over present methods may be accomplished.

# THE ECONOMIC SIGNIFICANCE OF SPECIFICATIONS FOR MATERIALS

## INTRODUCTION

MR. K. G. MACKENZIE[1] (*President, American Society for Testing Materials*).—The session this evening is being held under the joint auspices of our Society and the Western Society of Engineers, and it gives me great pleasure to extend to the members of the latter society a very cordial welcome. The session will be in the nature of a Symposium on the Economic Significance of Specifications for Materials. The subject will be discussed from the viewpoint of both the producer and consumer of steel, the producer and consumer of concrete and from the viewpoint of the purchaser of materials in general. The discussion should bring out the value of the use of specifications by industry. The five formal papers to be presented are as follows:

"Value of Specifications in the Manufacture of Steel," by John Brunner, Manager, Department of Metallurgy and Inspection, Illinois Steel Co., Chicago, Ill.

"The Economic Significance of Specifications for Materials from the Standpoint of a User of Steel," by P. Parke, Chief Engineer, The Pullman Co., Chicago, Ill.

"The Economic Significance of Specifications for Materials from the Point of View of a Producer of Concrete," by J. P. H. Perry, Vice-President, Turner Construction Co., New York City.

"The Use of Specifications for Concrete from the Point of View of the Consumer," by Arthur R. Lord, President, Lord and Holinger, Civil and Architectural Engineers, Chicago, Ill.; Vice President, American Concrete Institute.

"Specifications from the Standpoint of a Large Purchaser of Engineering and Special Materials," by J. W. Bancker, Vice-President, Western Electric Co., New York City.

In addition to these papers the subject of the purchase of motor oils will be discussed by Mr. H. C. Mougey, Assistant Technical Director and Chief Chemist of the Research Laboratories of the General Motors Corporation.

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[1] Consulting Chemist, The Texas Company, New York City.

It now gives me great pleasure to introduce Mr. Frank D. Chase, President of the Western Society of Engineers who will formally open the symposium.

MR. FRANK D. CHASE[1] (*President, Western Society of Engineers*).—I am very happy to acknowledge the courtesy extended to the Western Society of Engineers to participate in the sessions of this annual meeting of the American Society for Testing Materials. We appreciate not only this courtesy but the opportunity given us to participate in the constructive work of this Society.

The work of the A.S.T.M., as evidenced by the proceedings of this annual meeting, has impressed me more than any engineering activity with which I have ever been in contact in my engineering career, and I consider it the most valuable technical work being done by any engineering organization in this country today.

The work of the Society is of incalculable value in two definite fields: First, increased safety of human life, and second, economies in industy.

The first, and to my mind, the most important, is being brought about by the standards and specifications of the A.S.T.M., the use of which lessens the number of fatal accidents due to improper materials. We do not have the failures with resultant fatalities that we used to have. We all remember the news items of years ago—"due to the failure"—"due to the collapse," etc., of materials or structures.

In the second field of value, namely, economies in industry, the A.S.T.M. plays an important rôle. This nation faces today, as never before, a period of intense industrial competition, both at home and abroad. It is a time of "the survival of the fittest"—and there is only one way in which we can survive, either as individuals or as a nation, and that way is by the introduction and practice of economies, in materials, methods and processes, to secure reductions in manufacturing or production costs. The work of the A.S.T.M. is directly focused to this end.

The work of the A.S.T.M. is directed wholly to the improvement of the tools with which the engineer works and is therefore increasing his efficiency, and his value to society.

I again thank you, Mr. President and your Society for the opportunity you have given the Western Society of Engineers to cooperate with you, and I hope that such cooperation may continue for many years to come.

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[1] Frank D. Chase, Inc., Engineers and Architects, Chicago, Ill.

PRESIDENT MACKENZIE.—I am sure we are all very grateful to Mr. Chase for being with us and, on behalf of the American Society for Testing Materials, I wish to express to him our appreciation.

I now come to the introduction of the chairman of the meeting. He is Dr. Zay Jeffries, Consulting Metallurgist of the Aluminum Company of America; Consultant, Incandescent Lamp Department of the General Electric Co.; and also Consultant, National Tube Co. I take great pleasure in presenting Doctor Jeffries.

CHAIRMAN ZAY JEFFRIES.[1]—This meeting is conceived along somewhat different lines from the orthodox A.S.T.M. meeting. The committee responsible for its arrangement consisted, in addition to the speaker, of G. E. Warren, Assistant General Manager of the Portland Cement Association, E. S. Nethercut, Director of the Western Society of Engineers, and C. L. Warwick, Secretary-Treasurer of the A.S.T.M. This committee decided, that inasmuch as the Society had been busy for years manufacturing specifications, it would be well to get some idea as to how well the specifications were selling. They had heard that certain factories in this country, I do not know just which ones, had been able to manufacture products faster than the rest of the world felt like consuming them. I do not know where they got their information, but we will assume that it is authentic. They thought perhaps the A.S.T.M. might either be in the same position or might get in that position in the future, unless reports from the firing line were received. Consequently, the present session was planned. If the session tonight is somewhat different from most of the other sessions of the A.S.T.M. such differences merely reflect the influence of the Western Society of Engineers.

The broad subject is the Economic Significance of Specifications for Materials. We all know that specifications have an economic significance, or a society like the A.S.T.M. could not have lived for some twenty-nine years and have grown continuously during its lifetime, or various of the larger corporations would not have maintained standards departments for the very purpose of making specifications for their own use. We want, however, to get the viewpoint of the producers and the consumers at least in two fields of activity. The matter of specifications is something which will be with us more and more as time goes on. We need a shorthand method of harmonizing the activities of diversified groups, and we need a shorthand method of referring to things. Our civilization is getting so complex that we need to visualize volumes when certain words or phrases are mentioned. There has been a good deal of work done in times past in trying to define the word steel, and as the definition becomes more complete, it becomes longer, and many of us, at least, came to the conclusion that we are going backward when we try to define one word like steel with a few other words. The fact that the word steel is only one word is its main virtue, and when we use the word steel now, the term brings up to our minds various experiences. To certain men it brings up a lifetime of experiences. To certain others it brings up passing experiences. Or consider "Encyclopedia Brittanica"—the mere mention of the word calls to our minds the degree of familiarity which we have with those volumes. To one who is very familiar with the volumes the mere mention of the word calls to mind a great deal. To one who is not very familiar, it calls to mind very little. And so with the things with which we work day in and day out, it is not only permissible, but it is essential that we develop a shorthand method of expression in order that as little time as possible will be wasted with non-essentials. For example, in the field of specifications we all know of one of the series of names of one of our sister societies, the Society of Automotive Engineers, and the mere mention of S.A.E. 1010 or S.A.E. 1020 brings to our minds a tremendous number of things regarding those particular steels, and yet the mention of the term S.A.E. 1010 is sufficient as a shorthand vehicle to converse quickly and accurately about that material. So, as time goes on, we will be compelled to have more and more to do with specifications and we will find it necessary to use the specifications by numbers and names, and therefore it will be more and more necessary that we know these specifications by numbers and names, so that we can transact our business the more quickly.

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[1] Consulting Metallurgist, Aluminum Company of America, Cleveland, Ohio.

Two materials fields were chosen, namely, steel and concrete, together with a discussion from the standpoint of a director of purchases. We shall now proceed with the presentation of these five formal papers, designed to introduce this discussion.

# VALUE OF SPECIFICATIONS IN THE MANUFACTURE OF STEEL

## BY JOHN BRUNNER[1]

With the development of the bessemer and especially the open-hearth processes of making steel in large quantities, and at a reduced cost, its use has grown to such an extent that it now materially affects our mode of living and tends to influence the development of modern civilization in many directions. Through its use, marked changes have been made in all fields of construction, manufacturing, agriculture, transportation, preservation and distribution of food supplies, and of many other commodities, making the steel industry one of the most important basic industries.

With such an extensive application of steel, many different requirements are placed upon it so that it may be of a quality that properly functions in service. The many years of experience and observation in its manufacture and in its performance in service have enabled engineers to prepare specifications for the various qualities required in its many applications. The number of varieties of steel in respect to special qualities required by the users have steadily increased until today the steel maker has a large number of different specifications to comply with in manufacturing steel.

It is apparent that in order to furnish the proper qualities, the steel maker must be closely guided by requirements of specifications and, hence, the specifications are of great value to him in his operations. Many specifications, like those of the American Society for Testing Materials, are the result of cooperation between the representatives of the users and of the producers of steel and are used to a larger extent than other specifications in connection with the grades they cover. Many other technical bodies have gone into the matter of standardizing specifications, and, hence, we have today a large number of standard specifications covering many grades and qualities of steel.

Under some specifications, the steel is tested, inspected, and accepted before it leaves the mills. In other cases, where the specifications relate to chemical composition only, and the tests are made by the users, the proper interpretation of requirements as to other qualities in the steel is often obtained through personal contact between the user's and producer's technical staffs and through observation of its performance at the user's plants.

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[1] Manager, Department of Metallurgy and Inspection, Illinois Steel Co., Chicago, Ill.

With the close competition which exists between the fabricators in selling their finished products, the performance of the steel in their operations as far as it affects their final cost and the quality of their finished products, becomes most important. The number of operations used by the fabricators in the production of their finished product are usually reduced to a minimum within the working endurance of the steel in order to obtain economic fabricating costs. Hence, the steel maker must at all times exercise the greatest care in his production and be in touch with the performance of the steel in the fabricator's plants so as to develop a steel which meets the requirements in connection with his particular equipment and method of manufacturing.

While specifications cover the principal requirements of the quality of the steel as determined by tests, the question of furnishing a steel best suited for the particular purpose for which it is to be utilized, in the last analysis, rests with the steel maker, as specifications cannot be so complete as to fully prescribe the quality of the steel which would give the best performance in the further fabricating process and in service.

The steel maker must, therefore, select and use the proper kind of raw materials in the right proportions, the proper time in each of his processes, the proper temperatures in each process, and many other details which are required to produce the best steel for a given purpose. These details cannot be covered in specifications nor can they always be closely checked by tests made on the final product of the steel mill.

With the rapid development in the use of steel, an increasing number of technical men are employed in the steel industry, and these men are engaged in an earnest study of developments and of improvements in manufacturing methods and in the final quality of the steel products sold to the ultimate consumer.

With the research, investigation, and effort now being made to obtain the best results in the final products, the specifications for steel have increased both in number and in their requirements in order to cover the developments and secure the improvements desired. As a result of these efforts, many different compositions and alloys are employed by the fabricators to meet their particular aim.

The steel maker has more specifications to comply with today than he has had at any time in the past. A few of these specifications apply to special structures or special purposes only and cannot, for this reason, be standardized. There are many others which could be standardized in groups, resulting in the best economy in the production and in the use of the steel.

Specifications which accompany orders for steel may be divided into several groups in accordance with their requirements. Some specifications, as those for structural materials, etc., require physical tests on specimens cut from the finished product at the steel maker's plant, and the results of these tests form the basis of acceptance. This group of specifications is, in general, well standardized with definite and economic requirements.

Another specification group requires chemical composition of the steel together with impact tests. Railroad rails and railroad axles come under this group in general, and the product is accepted on the tests at the mill. This group covers highly important grades of steel and is well standardized with slight variations to cover local conditions.

Another group requires that check analyses shall meet certain requirements as to composition. Attempts have been made to standardize this group but standards have not become general in use.

A small group of specifications requires that physical tests as well as chemical composition fall within certain ranges. It is often difficult to meet the requirements in this group as the two sets of requirements are not always properly coordinated with each other.

Other groups require hardness tests, fracture tests, deep etch tests, grain size tests, and cleanliness tests. No definite standards have been established for most of these groups.

One of the groups, and one which covers considerable tonnage. accepts the steel on analyses made on test ingots taken at the time the steel is poured.

Many specifications, before they are transmitted to the mill for execution, must include not only quality requirements, but also rolling tolerances, cutting tolerances, branding, stamping, tagging, bundling, blocking in the cars, and other details which cannot be covered by standard specifications but must be developed for each customer to suit the unloading, handling, and manufacturing equipment which he has in his plant.

All specifications relating to orders in the steel manufacturer's plant are distributed to the men in responsible charge of any part of the steel making, rolling, etc., and serve as guides throughout the various processes so that when the steel reaches the inspection, it will comply with the requirements of the specifications. Some of the steel is inspected by representatives of the purchaser but a large part of it is inspected by the steel maker's inspectors, in which case the steel maker's responsibility, as to compliance with the specifications, usually does not end until the material has been used by the fabricator. This illustrates the value of specifications to the steel maker and the necessity of having complete specifications for all steel products in order to satisfactorily and economically fill the orders.

In the requirements of specifications the necessity of sufficient tolerances and ranges in quality, as well as in section and size, should be recognized so as to permit the steel maker to conduct his operations consistent with good practice and in line with the best economy, as it is reflected on the product furnished the ultimate consumer.

Through the cooperation between steel users and steel producers, improvements in the steel quality required for the particular product in which it is used have gone forward at a rapid rate during the past few years. To accomplish this, new equipment, new methods, and new practices have been evolved, but we have evidently not reached the limit as yet, and better accomplishments may be obtained in the future.

Steel, for a long time to come, will continue to be the most reliable construction material in most fields, and its use will increase as new fields, in which it can be used, are developed.

The American Society for Testing Materials has done splendid work in standardizing specifications, and several other technical organizations have followed its lead, thereby obtaining better material at reduced costs. However, the work of standardization has only begun, and it should be continued with all vigor to accomplish its aim, namely, the greatest economy to the ultimate consumer.

# THE ECONOMIC SIGNIFICANCE OF SPECIFICATIONS FOR MATERIALS FROM THE STANDPOINT OF A USER OF STEEL

BY P. PARKE[1]

The organization with which I am connected has a plant capacity to consume over 150 carloads of ferrous metals per day, or 2,250,000 tons per year. Appreciating as I do the compliment of the invitation to speak on the subject assigned, I assume that I owe the invitation to the fact that I represent such a potential consuming capacity and that my observations may be expected to reflect experience in dealing with materials on rather a large scale. At the outset I have to make acknowledgment of valuable aids which the American Society for Testing Materials has rendered to industry through the development of standard material specifications. Of these I shall attempt briefly to point out the why and wherefore, with special reference to the uses of steel; to suggest the useful purposes which standard specifications serve and the manner in which they are of economic value to the user. It has seemed to me that, just as insistence on accurate definitions gave birth to philosophy, and philosophy in turn flowered into pure science—so the trend today is toward the crystallization of more knowledge and the elimination of mystery concerning the things with which we deal in everyday life. What is the demand for a specification, but a repetition of the age-old desire for a definition of the needs of man? In its economic significance, the specification is an aid to the progress and well-being of mankind.

In general, material specifications are of particular economic importance to the large user of steel in that:

1. They are evidences of the fact that thought and study have been given to the service requirements for which a particular material is intended.
2. They constitute a standard for measuring and checking up on materials as supplied. Such checking insures against delay and waste in fabricating plants which would occur with improper materials.
3. They are invaluable to the large consumer who commonly draws his material from several sources of supply. Their proper application in such cases insures that materials from different sources will be of uniform and suitable quality.

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4. They promote the highest and best use of materials for each particular purpose.

5. They open a field of fair competition.

Taking up these five points in order, it is obvious that when a purchaser orders materials to a specification, whether it be his own, the manufacturer's, or those prepared by a technical society such as this, he has studied his requirements and concluded what materials are most suitable for his purpose. In ordering to specification, he benefits from all the research, experimental and development work of which the specification is the final summary. The question may be asked, "Why so many sources for specifications? why not buy to the manufacturer's specification?" The answer to the latter part of the question is that this is frequently done; and to the first part, the answer is that the individual manufacturer's specification commonly represents the product which he can most conveniently produce. The manufacturer's specification is valuable as a guide to the trade, though it may not precisely comply with the requirements of the buyer. Specifications supplied by an organization such as this have a greater value because they represent the results of the composite effort of the various interests making up the trade as a whole. No doubt, in many cases the specifications of this Society are compromises, but in all cases they are reliable and are a valuable guide to consumers. But many users have problems of their own, necessitating studies and investigations which may indicate certain modifications or deviations from standard specifications. The usual procedure in such cases is to formulate, on the substantial foundation already laid, the user's own specifications. As experience shows the way, he revises his specifications and keeps them up to date. The thought, study, and compilation of experience incident to this process must inevitably be of value alike to the manufacturer, the user and the industry as a whole.

As to my second point, that specifications constitute a standard, a yardstick, for checking up on materials. All products of one industry, as they become the materials of another, must be carefully checked. The user of steel must know that the material supplied to him is precisely what he wants so that it can be put into production without hitch or hesitation. If an error is discovered after production starts, the result will be a shop tie-up, with machines idle and production schedule disjointed until replacement is made. With improper material, a costly amount of scrap must result. Without specifications there are too many chances for something of this sort to go wrong. The purchaser may leave the matter of specification to the producer, who will do his best to supply precisely what is wanted, for in such cases he commonly guarantees his product. But something may go wrong either through lack of complete understanding or through error in the producing plant. In that event the user, not having ordered to specification, must rely on the manufacturer and has no means of checking the material before starting production. Improper material will cause delay and expense to the user, because while the manufacturer replaces defective material, he does not stand the consequent expense incidental to delay. In referring to improper material, I may, by way of illustration, refer to such items as sheets and plates. These may be perfectly proper for certain uses, but undesirable for others. One user may emphasize strength, another ductility for cold pressing steel; yet another may require a special surface finish and uniform thickness. Again, in forging steel, one user may require the maximum degree of ductility for materials subject to shocks; another may need the maximum strength where stiffness is important; and yet another may find that the cheapest grade of material will answer his purpose, and that to use anything more expensive would add to his cost without in any wise improving his product. A specification to cover the precise requirements is the best and only assurance of obtaining entirely suitable material.

Taking up my third point, namely, that the proper use of specifications insures uniform quality of materials from different sources, it must be apparent that specifications are of particular importance. A user of steel may require deliveries at widely separated localities but of absolutely uniform quality. If, for example, we require steel wheels delivered at the Atlantic seaboard, we do not order them from Chicago; yet we must be sure that wheels, from whatever source, shall conform to a rigidly fixed and uniform standard of quality. Both safety and economy in wheel service depend upon the closest adherence to the standards which have been developed and proved by service. It is highly improbable that anyone in a position of responsibility would purchase or use in passenger service, steel wheels from a variety of sources on the manufacturer's guarantees, without specification to insure uniformity. All manufacturers have not had the direct service experience so essential to safety and economy in wheel operation. Only when the cumulative experience of wheel-makers and wheel-users is embodied in a specification, can uniformity be insured among wheels drawn from a variety of sources.

I have said that specifications promote the highest and best use of materials. To this end, of course, a specification fully outlining the requirements is essential. The user from his experience with various grades is in excellent position to tell what grade of material will answer his purpose. There comes a time when special analysis and thought on the most economic use of materials is of first importance. Thus we may ask, "Are we getting all we can out of carbon steel? Can or should alloy steel be substituted?" In asking these questions we may have in mind considerations of strength, or safety, or of economy. When it comes to the soundest economic use of materials, the employment of specification in inquiries for material not infrequently leads to helpful suggestions from manufacturers. Their experience may have enabled them to suggest certain changes in specifications through which costs may be reduced. Thus the highest and best results economically will be obtained from sheet steel when no useless finishing labor is expended on it. A high degree of finish should be used only where it is essential; plain rolled mill finish should be used where it will answer as well. At times the highest and best use of sheet steel may justify the use of a costlier alloy product; and the principle of best use is applicable to other varieties of steel. But as to all steels, and all uses of them, it is a safe generalization that the highest and best use will be promoted by a full knowledge of the possibilities of the product as embodied in complete specifications.

Finally, I come to my proposition that the use of specifications opens a field of fair competition. If it is true that competition is the life of trade, then it seems to me equally true that only by the use of specifications can a fair competitive field be insured. How else can a user make known his precise requirements to competing manufacturers in such a way as to insure that they will all have precisely the same understanding of his needs. It simply cannot be done. To use steel wheels once more as an illustration, I may note that wheels are a highly competitive product; but they are such, only because of the use of specifications. If any steel of special quality is needed, a specification showing the requirement immediately opens up a competitive field.

In conclusion, as one of those who in the workaday processes of industry have learned how helpful is the work done by such organizations as the American Society for Testing Materials, I cannot take my leave without a word in acknowledgment of my own debt. The activities of this Society have accomplished much in committing the country's industries to policies, programs and methods which are in the best sense cooperative. They make for the larger interests of both manufacturers and users of all our products. By just such processes, I am sure, we shall continue to bring into American industry an efficiency which will insure the fullest benefits from increasing standardization, while avoiding any tendency to decrease initiative, discourage inventiveness, or atrophy that play of the imagination which is at last the very essence of inspiration for progress.

# THE ECONOMIC SIGNIFICANCE OF SPECIFICATIONS FOR MATERIALS FROM THE POINT OF VIEW OF A PRODUCER OF CONCRETE

By J. P. H. Perry[1]

I have been asked to discuss the "economic significance of specifications for materials" from the point of view of a producer of concrete. It is difficult, without appearing to be trite, to discuss this subject at a joint meeting of the American Society for Testing Materials and the Western Society of Engineers whose members probably realize more fully than any other group of people the real economic significance of specifications for materials.

Today, due to highly competitive markets and the unsound urge upon many producers to take business at any price, often regardless of actual cost, the economic significance of specifications for materials is of especially vital importance to the three major parties concerned —consumer, purchaser and producer.

Generally speaking, a specification under which any material is to be purchased should be rigid enough to fully cover the requirements of the work but should not contain unnecessary restrictions which will increase the cost or restrict the supply. The specification should be a complete and definite statement and, above all, it should be brief. I make this last comment feelingly because only a few days ago I attempted to wade through a specification that was by actual measurement over 3 in. thick. It could have accomplished its purpose admirably and still not have been over $\frac{1}{2}$ in. thick.

A properly written specification will have only the proper effect on the cost of materials. On the other hand, a poorly written specification will have a poor effect on the cost of materials.

In the invitation to address you which I received there were six questions outlined covering points which it was hoped this meeting would discuss. Before taking these up specifically from the point of view of a producer of concrete, I should like for a moment to discuss concrete and the factors affecting its cost in order to refresh the minds of many of you as to what a specification for concrete materials should cover.

First, let me say that in the discussion which follows I have assumed that the term "concrete" could be taken to include reinforced concrete as well as plain or mass concrete. Further, I am going to discuss concrete and reinforced concrete as they are generally used in building construction.

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What is concrete? The Portland Cement Association's book on "Design and Control of Concrete Mixtures" describes it as follows:

> Concrete is a mixture of cement, water and inert materials, put in place in a plastic condition, but hardening soon after, due to the process known as the hydration of the cement. Unlike most other materials, because it is placed in this plastic condition, concrete cannot be tested for quality at the time it is fabricated. This imposes severe restrictions on the making and placing, for unless each of these is carefully carried out, together with the subsequent curing, the finished product may not meet the requirements of the work.
>
> The fundamental requirements of practically all concrete are strength, durability and economy. These can be obtained only by proper selection of the materials, an intelligent design of the mixture and the adoption of proper methods of mixing and placing the concrete and protection during the curing period. The effect of each of these factors has been definitely determined by extensive research, the results of which make it practicable to design mixtures so that the properties of the concrete can be predicated at the time of fabrication with considerable accuracy.

The consumer and purchaser of concrete is interested in the finished hardened concrete ready for use, and in its cost.

In order to study the effect of specifications for concrete on its cost, let us consider the factors that determine the cost of concrete ready for use:

1. The cost of any plastic concrete as it leaves the mixer will be determined by the cost of the raw materials in the mixer and the cost of mixing.

(*a*) The cost of the materials in the mixer will be determined by the amount, kind and cost in the mixer of each of the ingredients required for a particular concrete. The cost of each ingredient will be determined by its cost delivered to the job, plus the cost of testing it, receiving it, protecting it, and placing it in the mixer. Raw materials going into concrete consist usually of:

Cement;
Water;
Aggregates;
Other ingredients like waterproofing compounds, etc.

(*b*) The cost of mixing, generally speaking, will be determined by the time required to mix the contents of the mixer, power costs, volume of concrete to be produced in any particular time to meet job requirements, and the mixing plant layout.

2. The cost of any concrete after leaving the mixer until it is in place will be determined by the following factors:

Cost of distributing;
Placing;
Tamping;
Screeding;
Size of pours;
Frequency and type of construction joints;
Characteristics of the structure itself;
Amount and density of work to be built into the structure monolithic with the concrete work such as reinforcing steel, conduits, inserts, sleeves, etc.;
Finish as affecting the placing of the plastic concrete;
Protection;
Forms.

3. Finally, the cost of hardened concrete ready for use after being mixed and placed and hardened will be determined by the following:

Stripping or removal of forms;
Continued protection;
Curing;
Pointing and cleaning;
Final finish.

You may say as you reflect on this long list of factors affecting the characteristics as well as the cost of concrete that concrete is a rather complicated material. It is, but no more so than many other structural materials, for today due to painstaking and thorough research both in the laboratory and in the field and from years of experience in producing this material for great numbers of structures, the industry has advanced to the point where with skill and knowledge *it knows just how to produce with assurance* any particular kind of concrete having definite characteristics. Concrete today can be considered a standardized and stable material. In our larger cities, plastic concrete is being produced in central mixing plants and sold to the trade ready for placing.

All of the many factors noted as affecting the characteristics and cost of concrete ready for use can be evaluated today with assurance. The one prime factor important above all others in determining all these factors is the use to which a particular concrete is to be put. "*Use*"—answer this for any particular structure and all the factors listed can be determined and evaluated.

The consumer or purchaser having decided on this "use" factor can, with the knowledge and data available, specify briefly or extensively all that the producer needs to know in order to produce the concrete desired and to determine its cost.

Such a specification prepared in the light of our present-day knowledge of concrete and concrete structures should be a specification that will have only a favorable economic effect on the cost of a particular concrete. It has been definitely established that good concrete produced according to the latest developments in the art is economical concrete both for the consumer and the producer.

As you see, a great many factors affect the characteristics and cost of concrete ready for use, and it is not possible, in the time allowed, to discuss in great detail these many factors. I can only discuss them in general and point out but a few things in detail.

Now to discuss the six questions previously mentioned as the points to be discussed in this meeting.

### *I. What are the advantages and disadvantages of specifications to the producer of concrete?*

I have emphasized the fact that the specification writer should keep before him the use to which the concrete is to be put. I should like also to emphasize almost equally strongly the fact that the specification writer should reflect on the use to which the specifications are to be put. Broadly speaking, specifications have three major functions, that is, from the point of view of the producer of concrete, which is commonly the point of view of a contractor or builder. These are:

(*a*) As a basis for estimating or bidding the job.

(*b*) As a guide to the operating or construction forces in building the work.

(*c*) As a basis for paying for the work and for settling any disputes.

A study of the factors which influence the cost and quality of concrete invite the thought that a specification is necessary and therefore must be advantageous. We have seen that an adequate specification should be written with a knowledge of the use to which the concrete is to be put, together with a knowledge of the present-day developments in the art of making and placing it; a knowledge of the limits of accuracy to be economically obtained in its use; a knowledge of the local supply of concrete aggregates; a knowledge of the requirements as to making, placing and stripping of forms; and a knowledge of the finishing and curing requirements—all of which knowledge is readily available.

I have used the word "adequate" specification. The specifications as they come to the average builder of structures, involving concrete and reinforced concrete, are good, bad and indifferent. Sometimes one is tempted to classify them, having in mind Amos and Andy and the frequent answer they give "Yes and no, but mostly no." The good specification is the exception rather than the rule. The good specification, however, enables a producer to fix with assurance the cost of the concrete required and it enables him to make the concrete as the consumer wants it. The responsibility is clear. The good specification enables the contractor to prepare his estimates and bid properly on a given piece of work.

There is never much time allowed for estimating work. When the specifications are poor there results unsound bidding. This usually results further in someone getting the work and losing money or skimping the job in an effort to avoid losing money. The really capable producer frequently loses the job and the consumer is thereby a further loser in that he may have to fight all the time, which results in excessive inspection, or administrative costs, or in ultimately the consumer accepting faulty work with consequent higher maintenance expenses, or even graver losses. All of this is a direct economic loss.

Even, however, granting that a satisfactory producer of concrete can be secured as a result of a competition held on unsatisfactory specifications, we still face the economic advantage or disadvatage of good or bad specifications growing out of the operating of the job or the erection of the work. Unquestionably better workmanship, more orderly progress and lower costs result from sound, clear, orderly, intelligent specifications that are accurate and definite, than can ever result from loose, unfair, verbose, unintelligent specifications. Nothing takes the morale out of the job more than indefiniteness as to what is to be done. When a superintendent has to keep calling up his general superintendent and the latter in turn bothering the architect or the owner's engineer for interpretation, something is wrong.

Granting that the job can be let and be built from unsatisfactory specifications, there is the further economic advantage or disadvantage in the settling up and paying for the job and the possibility of having to settle disputes. If the specifications are clear and good and sound these potentialities are minimized.

Today there should be no excuse for anything but good specifications, so far as concrete and reinforced concrete are concerned. The concrete industry has set up well-established sources of information, readily available to any consumer, so that proper specifications may be written, either at length or in abbreviated form. Producers today are also increasingly willing to cooperate with purchasers and consumers in the writing of specifications to see that they are consistent with economy, as well as with current practice and with good workmanship and quality of materials.

Today, by requiring that the standard specifications of the Portland Cement Association, the American Concrete Institute and the American Society for Testing Materials be followed exactly, *the consumer can purchase satisfactory concrete by specifying only the water-cement ratio and stating that aggregates shall be such as to produce a plastic and workable concrete.*

The vast majority of specifications for concrete and reinforced concrete would be greatly improved if they were about one-tenth their present length, and fundamentally referred to the standard specifications of the three institutions just named.

The foregoing gives a general outline of the ideal of good specifications. It might not be inappropriate to mention a few specific things that we men in the building industry so frequently find in specifications for concrete work today which cause unnecessary difficulty in executing work.

1. We do not find that sufficient attention is paid to the size and grading of aggregates economically available in a given locality. It is quite customary for architects and engineers to specify in detail the size of gravel or crushed stone which should be used without considering just what is available.

2. We find there is a tendency to keep the size of coarse aggregates too small. There is no use in specifying a $\frac{3}{4}$-in. maximum size when say, $1\frac{1}{4}$ in. would be satisfactory, for within certain limits the coarser aggregates will yield a more economical concrete.

3. We still find that the old volume proportions, for example, a mix of 1:2:4, are given, in addition to specifying strength and water-cement ratio. This is confusing and costly. Code requirements may make it necessary to adhere to the old volumetric proportions. If so, specify them only. However, generally speaking, today if one specifies the water-cement ratio and states that aggregates shall be added to produce plastic and workable concrete, he has said enough. Give the producer a chance to select aggregates of suitable size and grading.

4. In regard to admixtures, such as waterproofing compounds, specifications are not clear. It is not quite fair that an architect or engineer should specify that concrete shall be waterproofed or made water-tight by the admixture of a "satisfactory waterproofing compound." The specifications should state that the concrete should be made more impervious by the addition of a "definite waterproofing compound." Take time to specify definitely the material you want used.

5. Consider carefully the time of mixing specified. It is not necessary for the regular run of concrete work to call for more than a minute mixing time; at times $\frac{3}{4}$ minute is enough. However, if some particular concrete must meet a particular condition, be sure to specify clearly just how long it shall be mixed.

6. For large and important work in outlying districts, where producers generally do not have definite information on available aggregates, it would be advisable for the architect or engineer to examine and test these available aggregates, and specify the proportions to be used, together with the water-cement ratio. This will result in savings to the consumer. If this information is not available, the producer has to guess on the safe side, or go to the expense of making these tests, which time does not usually permit of his doing.

7. We still find a lot of unnecessary detail limiting the making of forms, the time to be allowed for stripping, etc. For instance, a specification just received states that "forms shall not be removed until directed by the architect," and then the next section states that "forms may be removed under certain conditions within 24 hours," without specifying the conditions. Just how shall the producer proceed to evaluate this item?

A great deal more might properly be said in a meeting of this kind on the way specifications are usually witten, covering the placing and removal of forms and the length of time they must remain in place. On private work I should like to suggest to specification writers of the future that they go as far as specific administrative conditions will permit them in leaving it entirely up to the contractor as to how he shall erect his forms, how long a period they shall remain in place and how they shall be taken down, inserting clear language definitely placing upon the contractor the responsibility for the safety of the structure. On public work and on some private jobs this latitude perhaps cannot be given. This is especially true in all situations where unlimited competition has to be received.

8. In regard to finishing and curing concrete, give some real thought to describing this. It is fairly expensive and important and it is not clearly covered in a great majority of specifications. As for example, I remember as though it were yesterday a case on a half million dollar bakery building. The architect, in the following language attempted to describe how the concrete was to be finished:

> The finish of all concrete work exposed to view, namely, ceilings, lintels, beams, girders, columns and partitions, shall be extra smooth and even. As soon as the forms are removed and the concrete is still green small defects shall be patched up. All uneven surfaces and other defects shall be scoured away with carborundum stone so that the entire surface is perfectly level and of even color.

Note the frequent use of the words "all," "even" and "perfectly." Interpreting this specification literally added $30,000 to one of the bids which the architect received. It so happened that that contractor was favorably considered by the owner and the architect and his estimate was discussed with him. After wasting a lot of words in trying to reach a meeting of minds as to what kind of exposed surfaces the owners and architect really wanted, a visit to one of the other bakeries of the owner was made. The language in the specification was changed so that all that was said about surface treatment of exposed concrete was that the workmanship and appearance should be equal to that of the building visited. The $30,000 item was eliminated and as a matter of fact the allowance which the contractor would have normally included in his estimate was partly saved.

There is a story told of the elder Pierpont Morgan as follows:

> After years of searching he located a professor of English who had the rare gift of selecting words which meant only one thing. Mr. Morgan employed this man at a large salary for many years to write the cables which his firm had so frequently to send while he was buying art objects in Europe. Mr. Morgan knew that when he received a cable it meant just what it was supposed to mean and nothing else.

If many architects and engineers offices would but hire professors of English with this same gift, the way of the contractor would be made much smoother.

In many cases where it is difficult to specify things which are fundamentally aesthetic, or matters of taste or judgement like smoothness of finish, not only for walls and ceilings, but for floors as well, it might be fairer for the specification writer to include a definite money allowance rather than to try and write English which is Amos and Andyish.

9. In regard to reinforcing steel, this is almost always shown on the drawings, and specifications are usually quite clear. However, we do find specifications calling for excessive tying of reinforcing steel. This is expensive and oftentimes not necessary.

10. If a great deal of work is to be built monolithic with concrete, some note should be made of this in the concrete specifications because it affects greatly the cost of placing concrete and sometimes makes necessary the building of more form work to provide additional working space for the different trades. There are certain noted concerns in this country who use the word "level," or "perfectly straight," or "absolutely true" and mean what they say—mathematically and legally, as distinct from what 99 out of a 100 interpreters of specifications would expect. I know of cases where inserts set in concrete

floors or curb angles set on shipping platforms have been lined up with the transit as though they would be used for some astronomical observations. Specification language is a serious thing.

11. In testing concrete we find that job test cylinders very seldom give the correct indication of strength of the concrete in the structure; in fact core samples from a structure usually test stronger than the job or laboratory test cylinders. This observation might be worthy of the average specification writer's further consideration as to whether he might not modify the present-day requirements.

*II. How are specifications being usefully applied in the production of concrete?*

Specifications are being usefully applied every day in the production of concrete.

A specification is referred to in determining the cement to be used; selection of aggregates; water-cement ratio; admixture to be used; time of mixing; determining placing requirements; construction and removal of forms; protection, curing and finishing.

Specifications are being used; the extent of their usefulness and helpfulness, however, depends altogether on the specifications.

Good specifications tend to increase generally the efficiency and quality of the output of any industry. As an example, just see what good specifications, prepared by all interested parties, acting together, have done to produce cement and steel products, such as structural steel, steel sash and windows, cement blocks, etc., of practically uniform quality in this country.

*III. What are the relative merits of purchasing materials to specification under test by consumer, and purchasing them by trade mark or trade name under guaranty from the manufacturer without test?*

Concrete is bought almost entirely under specification and test by the consumer. Trade-marked concrete is not usual. However, there are a few producers from whom a consumer could purchase a so-called trade-marked concrete with assurance. By this I mean, if the buildings or structures are built by contractors of assured reputation and long-time experience their pride in their craftsmanship functions essentially as a trade mark and great latitude can be given these firms when it comes to writing specifications. I might mention, however, that where central mixing plants are in existence, so-called certified concrete can be purchased—certified as to strength and consistency.

*IV. What are the economic benefits of standard specifications as applied to production of concrete?*

It would indeed be hard to enumerate all the benefits of standard specifications to the production of concrete. They are great in number.

Take cement: it is almost universally bought under the standard specifications of the American Society for Testing Materials. Reinforcing steel is also bought under the Society's standard specifications.

Aggregates are tested and graded in accordance with the Society's recommendations.

Concrete consistency is determined by the "slump test" as set forth by the Society's standard method of test. The concrete producer refers constantly to the standard specifications of the Society. They have become almost universal in use, and the savings resulting therefrom run into large sums of money each year.

Some authorities have said that standard specifications retard the development or improvement of materials. I do not think this so, if we keep in mind that standard specifications need to be revised as improvements in science and the art of making materials progresses. We should look at standard specifications as the *foundation* from which advances and improvements proceed. Persons who insist that the standardization of specifications retards the development of any material or art may have been bitten by the bug which has hit so many Americans, a bug which objects to the functioning of the time element.

We are too prone to rush from one idea to another. The very fact of having a standard interjects a period of time or experience which must elapse before the new idea can be taken up. Of course, standard specifications must be modified. That this is being done is evidenced by the American Society for Testing Materials revising its specifications as developments warrant. It is further evidenced by the fact that the building codes of the major cities of this country, difficult as they are to modify, are changed every few years to take advantage of advances in the art of building construction.

Industry as a whole, I feel sure, realizes the benefits of standard specifications. President Hoover, while Secretary of Commerce, initiated work along this line, and in New York City the New York Building Congress has successfully completed a standard specification.

*V. What trends are observed in the application of specifications?*

We find specification writers recognizing more and more the accepted standards. This is indeed a welcome sign, and it should continue.

We find a tendency for consumer and purchaser to consult each other in the preparation of their specifications. This should also be more wide-spread as it is a vital factor in effecting economy.

I think my observation, over a period of 25 years, and that of the responsible personnel of the company which I represent, would be that specifications are improving. There is still tremendous room for improvement but, so far as concrete and reinforced concrete are concerned, they are vastly better than even 10 years ago.

*VI. There is an element of cost in the application of specifications to purchasing materials for which there is presumably commensurate value received by the consumer. Are the benefits worth the cost?*

Yes, without question. Your being here tonight is the answer. All engineers and manufacturers recognize the value of specifications. One has only to look at the vast amount of time that has been spent in standardizing specifications and in stabilizing industry thereby. There are immense savings to be made through this sort of thing. Manufacturers can simplify and standardize equipment; workmen learn how to function with more skill; and we could go on at great length listing the benefits of standard specifications. In the concrete industry the data developed in recent years has helped to stabilize the production of concrete and the securing of a better product.

In conclusion, I would leave with you a few important points relative to specifications as effecting concrete and reinforced concrete.

1. We have been speaking as if specifications existed alone; on the contrary they usually are used in conjunction with drawings and are dependent upon inspection with its great variable of the human equation. I think really that one of the most frequent defects in specifications is that they are not properly coordinated with the drawings and that they overlook the nature of the inspection. These are both serious and should be allowed for.

2. Let us prepare our specifications seriously. They really put the breath of life into a set of drawings and make them real; and the drawings and specifications determine the cost of the work. The first impression an estimator forms in going through a set of drawings and specifications determines to a marked degree the kind of unit prices he applies to his quantities.

3. Be fair; place responsibility where it belongs. The writer of a specification should assume the responsibility for the proper choice of materials and not duck out under General Conditions, catch-all clauses, and "hide-in-the-dark" phrases. He knows just what the

consumer expects and the producer should only be responsible for making the thing required.

4. Arrange specifications systematically; keep things of a kind together; do not specify concrete work under some other part of the specifications or mix other classes of work in with concrete; take account of local trade practices and bear in mind the function of the subcontractor in practically all modern building work. The part subcontractors play on large buildings is apparently often not in the mind of the specification writer. The average big building is done from 80 to 90 per cent by sub-contract work. The work to be done by each of these sub-contractors should be absolutely segregated and specified completely together.

5. Use the knowledge, experience and data of the industry so easily available to those interested in preparing specifications. If possible, have close cooperation between consumer and producer in the preparation of a specification. It will save money for all concerned.

6. Consider the practical limitations of the material being specified. Do not expect concrete to look like cut stone—accept board marks and discoloration unless you want to go to the expense of special form linings and even then you still have the concrete as concrete but with a slightly different texture. Consider the degree of accuracy with which it can be economically wrought or used. Do not call for limits of accuracy or control which are not practical. The cost of this runs high and is wasteful. Do not expect granolithic finished concrete floors to be as level as a steel plane. In other words, consider trade tolerances as applying to concrete as much as they do to machine design.

7. Do not specify the results you want, and then specify in every little detail each step in the producing of the results, unless you want to pay more than is necessary for it and then often not get it.

8. Finally, any piece of work worthy of a good complete set of specifications is worthy of intelligent inspection by a competent inspector thoroughly familiar with the work. Nobody realizes the tremendous cost to a producer of poor, unintelligent inspection. It is a direct economic waste. The reputable, competent producer on the other hand welcomes competent inspection.

9. Above all be brief, say things once and only once. Have courage to take responsibility.

*Acknowledgment.*—In conclusion, I want particularly to acknowledge my obligation to E. K. Abberley, Engineer of Turner Construction Co., for assisting me in preparing this paper.

# THE USE OF SPECIFICATIONS FOR CONCRETE FROM THE POINT OF VIEW OF THE CONSUMER

By Arthur R. Lord[1]

I feel somewhat overwhelmed by the dignity of this occasion. It seems to me that a man is neither natural nor happy when engaged in being dignified. Besides, my subject this evening does not lend itself to such treatment. Concrete is still very young, robust, full of keen chemical spirits. Of the major structural materials with which the specification writer has to deal, concrete alone is *alive.* All the others are already "finished products" when he deals with them. Possibly this is the reason why concrete sometimes is considered the willful, bad boy of the construction industry. Such a boy, as you all know, has tremendous possibilities for mischief and equally surpassing potentialities for constructive achievement. While it may be more peaceful to deal with certain other materials, it is undeniably more stimulating, a lot more fun, to deal with concrete—and in the successful end, more satisfaction is experienced. If you are acquainted with David Grayson's delightful book, *Adventures in Contentment*, you may recall that Charles Baxter, the village cabinet maker, chose *curly* maple when working for his own enjoyment.

One naturally, in preparing for a talk like this, first looks up what has been written by others along the same line. Pursuing this theory, I consulted first that classic treatise by the eminent specification writer, Ellis Parker Butler, entitled "Pigs is Pigs." You may not catch the reasoning on first thought, but the similarity in situation is striking. If you recall the story, I feel sure that you will agree with me that Mr. Flannery, the Westcote express agent, would have suffered none of his subsequent engaging difficulties if he had realized at first, as he did later on, that there may be a difference between pigs—between *Guinea* pigs and *Irish* pigs, at least. Now *that*, I maintain, is the chief cause of our difficulties with concrete, of our complications in attempting to specify this material. If we but remember that, most decidedly, Concrete is *not* Concrete, but is rather a whole prolific family of construction materials—or, if you prefer, one material whose properties may be varied and directed over an extraordinarily wide range to serve a multitude of diverse purposes—if we remember this, our past difficulties may be recognized as mistreated opportunities. We have been passing up the real fun, the chief skill of the game.

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I am endeavoring to bring out *this*, the chief thought of my talk this evening, by a background of pictures on the screen. By this device I hope to make double use of my limited time, by giving you two messages simultaneously—one through your eyes, the other through your ears—to make you realize better that "concrete" is a most collective noun. It really takes little imagination to see that the needed combination of qualities in concrete will vary greatly with the purpose that the concrete is intended to serve. In concrete roads, for example, durability, surface toughness and tensile strength would seem to be most vital. In a factory, on the other hand, compressive strength, rigidity and resistance to fire are the important properties, while the weather exposure may be almost *nil*. For a dam we must have concrete that is watertight and in which volume changes are small. If this concrete can be manipulated so as to knit together eagerly across the construction joints, so much the better. For wharves and barges we require concrete that will resist the chemical action of sea water, while other uses demand high protective value against many other chemical exposures. For ships, and for the floors of a high building or of a bridge, a concrete of light weight is most economical, while for the counterweights of the same bridge an abnormally heavy concrete is indicated.

Exposed concrete and stucco bring a different set of requirements. Architects have satisfied themselves in many instances that the natural surface and texture of concrete, including the markings left by the forms, may be beautiful under proper control. Others will have none of it, demand that the concrete be skinned by chemical or mechanical means, or covered by paints or stains that it must be specially prepared to receive. Still others seek by means of a dash coat or a full layer of stucco, the wealth of rich color and interesting textures that may be thus secured. Radically different is the fundamental technique developed by John Earley for his "plastic mosaics," in which the aggregates of an exceptionally dense concrete are made to provide controlled color. By one set of controls we have a Bahai Temple or a Fountain of Time—by a different set we have a Gingerbread Castle, of concrete, to delight New Jersey children.

A vast total of concrete goes into the manufacture of products. Here again a new set of properties is emphasized. No one wishes to drive nails into a concrete dam—unless it be Professor Slater and his testing gang—but nailability is essential in the furring and partitions of a house. Heat and sound insulation become vitally important.

For the architectural facing of buildings a super-dense, non-crazing concrete must be made, possibly by the vibration process. We find producers delivering concrete for this purpose in which the strength regularly exceeds 10,000 lb. per sq. in., although such crushing strength is not at all needed—it is merely incidental to the satisfaction of other requirements. Much of this concrete must be made so as to facilitate dressing by machinery to give the desired textures.

In gardens and on the facades of public buildings we find sculptural concrete, involving the reduction of settlement and of shrinkage, in order to preserve fine lines and to avoid damage at undercuts. The ornamentation of buildings, both sculptural and architectural, is now frequently made of concrete placed at the same time as the structural concrete—but *not* in the same manner. The necessary changes in technique are readily made, but they must be specified. In some cases it is not necessary to so much as stop the mixer, but the specification writer must stop—and think—to make this possible.

The Fountain of Time, in Chicago on the Midway Plaisance, presents an even hundred of larger-than-life-size figures cast in place in concrete. Lorado Taft, its sculptor, tells us that this group is based on a couplet of Austin Dobson:

Time goes, you say? Ah, no!
Time stays, we go!

A pessimist might have advised Mr. Taft, in order to fulfill these lines perfectly, to have made Father Time of stone and the other ninety and nine of concrete. But Mr. Taft is no pessimist. In this embodiment both the *Time* and the *We* are of reinforced concrete, and, fortunately, both have stayed for a matter of nine years, both bid fair to remain with us indefinitely. It would have been sad had some careless specification writer, of the *Pigs-is-Pigs* persuasion, had that initial responsibility, but it seems safe to say now that other work of the same enduring qualities may be produced at any time by the same measures.

It will not be simple to draft standard specifications of the usual type to cover the different combinations of qualities required by the varied uses of concrete. It seems apparent that standard specifications as now drawn do not meet our greater needs at all adequately. In many respects concrete does not lend itself to the standardizer. It presents quite different and more difficult problems, demands greatly different treatment, from most of the inert materials with which this Society has dealt so successfully in the past.

This may be better appreciated if we sum up briefly some of the properties of concrete that may be desired in various degrees for various uses. Fortunately these properties are susceptible to fairly wide, controlled, variations. Some, however, are mutually contradictory and cannot be had in the same concrete. Principal qualities, subject to control in this manner, include: *Strength*—tensile, compressive and shearing; *Rigidity; Permeability*—all the way from a sieve to absolute watertightness; *Absorption* and *Adsorption; Volume Changes*, including settlement, shrinkage, moisture and temperature effects, extensibility and plastic flow; *Heat and Sound Insulation*—including accoustical control; *Resistance* to Fire, to Impact, to Abrasion, to Chemical Exposures and to Weather Exposure of varying intensities; *Bond* to embedded metal, and to various base or cover materials; *Color*—changing with the materials and with time; *Surface Texture*—from newest modern to the simulation of centuries of weathering; *Weight*—from less than 40 to more than 300 lb. per cu. ft.; *Nailability, Workability*, and other -abilities for special needs.

To achieve the best combination of properties for a particular use, with a proper consideration of economy, makes the writing of a fine and fully adequate concrete specification a work of art and of science—and often a laborious task. But the game is fascinating, we learn from each experience, and, as a correspondent from South Africa expressed it happily, "We try out the ideas of other people and ourselves, and sometimes get surprising results." The surprises are not all unpleasant nor disheartening, for concrete follows fixed laws no less than any other material—only there are many more laws involved in the case of concrete.

The very abundance of the data of research and experience that have been published about concrete is an embarrassment to the busy specification writer. Some one must take this accumulation and distill from it simple usable rules. Abrams has discovered the basic law that water-cement ratio governs the strength of concretes placed at workable consistencies, and we know that this law is intimately related to some of the other properties of concrete. But there still remain many considerations for which water-cement control is not an adequate answer.

One trouble is that we have too great a choice of materials from which to make our concrete. Nathan Johnson has counted the number of cement and aggregate particles that go to make up a cubic yard of concrete and has arrived at a figure that represents the possible number of different combinations among these particles—such a figure as even an astronomer would not fully comprehend. While interesting as a speculation, this is hardly necessary. There are a sufficient number of different materials-used-to-make-concrete to provide a satisfying diversity, without calling in the microscope or the air separator.

There is cement. This Society once had portland cement tied to a standard specification that held for over twenty years, but this active material finally squirmed loose in attempting to satisfy the demands of an ever-widening use. Now we are content to have at least two standards, although I suspect that these still leave a few of the present-day cements at large. Some of the manufacturers of admixtures for workability or for waterproofing have visioned a Paradise in which their particular material will be ground into the cement at the mill. Quite apart from this hope, it seems reasonable to predict that our transforming chemists will continue to suggest new types of cement, designed to secure certain properties in the concrete in fuller measure. The several special cements now on the market are but the forerunners of such an host. It would be well if this Society should standardize them before any more are born, predetermine their sex and character, rigidly limit their number and diversity—and so save the specification writer of the future a lot of agony.

And there are aggregates. Not satisfied with the bountiful provision of Nature in this field, Man has set out to make concrete aggregates of almost everything from slag to garbage. There should be no surprise that this is true. What aggressive business man could really understand that a hundred million cubic yards of concrete are used in this country in a normal year and not think how nice it would be to tuck away a half-yard—or a tablespoonful—of his product or by-product in each and every yard of concrete? Only the human stomach offers the business man a comparable opportunity! . . . These new products are perplexing, but some of them will be useful when we come to know them better. Societies such as the American Society for Testing Materials should perform the introduction and not leave the matter entirely to the salesman and the promoter.

And there is machinery. Machines for washing and grading aggregates scientifically at the plant, machines for segregating them during handling, machines for re-combining them at the mixing plant, devices for re-segregation on the way to the forms, and finally, most diverting of all, the human machines that are supposed to correct all the troubles occasioned by the others. And yet, our machinery is being perfected all the time and really causes the specification writer surprisingly little concern.

And there's the weather. As Mark Twain said, we all talk about it on the slightest provocation, but we do nothing about it. The concrete specification writer has to do something about the weather.

Curing temperature and moisture are as vital factors in concrete making as the water-cement ratio itself. The specifications must provide for the proper weather locally—the right temperature and the right, properly distributed rainfall—in accordance with an abundance of information about the effect of curing conditions upon the properties of concrete. In one experiment in connection with casting and curing methods, at Chautauqua, our concrete appeared to have attained an age of a thousand years during the few days that we let it remain in the forms. Mr. Bates suggests, too, that a sufficient intensity of super-tropical weather will render the cement colloids irreversible and so prevent corrosion.

Formwork and reinforcing steel are other matters that require attention and decision from specification writers. There are wood forms, steel forms, pre-cast concrete forms, plaster forms and glue molds. There are tight forms and purposely leaky forms. Even the kind of lumber, and the tooling of it, may affect the texture of the concrete and its color values. For ordinary work, the wise specification writer leaves these decisions to the contractor, but for exposed architectural or sculptural concrete the forms become a part of the design, to be rigidly controlled. The fabrication and placing of the reinforcing bars and shapes may likewise involve considerations that the specification writer may not safely entrust to a standard phrase or two.

This is far from a complete list of the variables that, under proper manipulation, make concrete with a wide range of desirable qualities readily available. There are several guiding factors that help the bewildered specification writer to improve this situation to his own ends. Most important, perhaps, is the free and friendly intercourse that has become traditional among cement users. Secrets are out of fashion. What one man learns, even by his failures, he hastens to broadcast for the benefit of the rest of us. Sometimes he starts to cackle when he is merely thinking of laying an egg. Any concrete user who is not engaged in "research"—if it be but a few cylinders cooking beside the stove in the job office—is hopelessly out of step with the times. We are most teachable—even if we do insist upon all being teachers in our turn.

This abundant production of data of very diverse merit requires an interpreter for its safe use, nor is one lacking. We have several of them, of which this Society is one, and they generally leave us a choice as to what we shall believe. As with concrete materials, so with concrete standardizers, many combinations are possible, resulting in varying standards as we pass from one society to another. Concrete is so widely used that every group of technical men considers concrete to be its special field, while the annoyance of the specification writer steadily mounts. For each group is considering concrete in the light of some special set of use requirements, without adequate recognition of the whole field of this material. It would seem that the work of the various societies interested in concrete could be coordinated much better than it now is, to produce definite and authoritative documents on which all may agree. This would eliminate much of the present waste and confusion, and relieve overworked individuals who now travel a constant round from one convention to the next, doing the best they can to promote uniformity. Admittedly, this is a somewhat delicate task, but it seems to me well worth the effort and the risk. The point of view toward concrete taken in this talk might be helpful in the negotiations.

As you have seen upon the screen photographs of concrete in many of its applications, you may have felt that we really must know a lot about concrete to have been able to use it so effectively and so extensively. It is true that no *student* of concrete need have any difficulty—except in the matter of time—in specifying and in securing fine concrete on his work of whatever type. But not all specification writers can be students and specialists in so far-reaching a field. The problem would be simple, if all a specification writer had to do was to recite the qualities required in his concrete for a particular use, and to leave their accomplishment to the contractor. But at present we do not have, in the case of several important properties of concrete, so much as a commonly accepted measuring stick to express our requirements. This is an immediate task for the societies dealing with concrete. We may look forward to the time when a measure for workability may be mentioned in a scientific manner, without emotion.

Some of our problems will be simplified with time. The mixing of concrete is rapidly passing to central mixing and batching plants, where better equipment and control should be secured. The teachings of past years are slowly sinking into the consciousness of field and office workers. . . . But time will also add new problems continually —new materials, new machines, new data of research. Old ideas, once unsuccessfully tried out, will prove to be workable under new conditions.

The goal of this Society—standardization of the commonly used materials—is a far distant prospect as far as concrete is concerned. The activity of so many other technical societies at the present time in dealing with concrete indicates that the progress of this Society has been too deliberate to meet the needs of the specification writer and user of concrete. Proceeding upon the Hibernian theory that *Concrete is Concrete*, it may be that the best that you can hope to do is to provide a rough outline map, to be filled in by the traveler from the experience of careful observers who have recently passed that way. I believe, however, that the best interests of this Society, and of the great public (outside its membership) that it serves, will gain markedly, if the work of the Society should be re-mapped and restudied, in cordial cooperation with the other societies, giving full recognition to the fact that concrete is a *family* of materials rather than a single definite material, and that no single group can possibly comprehend all the useful variations to which concrete is susceptible under proper and complete control.

# SPECIFICATIONS FROM THE STANDPOINT OF A LARGE PURCHASER OF ENGINEERING AND SPECIAL MATERIALS

By J. W. Bancker[1]

The Western Electric Company is the manufacturing, purchasing, and distributing organization for the Bell System. As such, we manufacture telephone central office equipment, subscribers' equipment, and lead-covered telephone cable and we purchase telephone supplies of all kinds from pins to poles. We distribute all of this manufactured and purchased product where needed through thirty-four distributing houses located strategically in the territories of the operating telephone companies of the System.

Our purchases include the telephone supplies referred to and the materials, supplies and equipment which we use in our manufacturing plants, our total purchases running between two hundred and three hundred million dollars annually.

Some idea of the diversification of our purchases may be obtained from the fact that we regularly buy for use in our manufacturing plants more than 25,000 different items of materials and supplies and 3500 items for direct use of the telephone companies. These materials are purchased from more than 15,000 suppliers located in every state of the union.

We have been advocates of specification control for a great many years both within our manufacturing plants and in the purchase of materials, supplies and equipment from others. There is a basic reason for this. Service to the telephone using public demands that the telephone plant must be in good operating condition at all times. To make this possible we must be sure of the quality of materials going into that plant and we can only be sure by developing, through engineering study, the characteristics and requirements to be met, covering these by specifications that permit of practicable production and then inspecting the product received under these specifications.

Material specifications are, in the main, simply definitions of the properties of the materials which the purchaser desires. They represent his best efforts to state in measurable terms those properties which are necessary for satisfactory use at the least cost consistent with the desired quality. In addition, they include methods of test and disposition of material in case of failure to meet requirements and serve to provide a basis of inspection mutually agreed upon between the supplier and the purchaser, and, therefore, they provide a basis for obtaining comparable competitive bids.

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Acceptance of a specification by the producer is a pledge to relative uniformity at a definite level of quality. The quality level and the degree of uniformity demanded by the specification should therefore be wisely proportioned to the needs of the consumer if the advantages of this method of purchase are to be secured.

The majority of the items we buy involving large purchase value or special quality considerations are now covered in this manner, and specifications for the other materials are being issued as fast as expediency and economy will permit. The noteworthy aspects of our material specifications are the great diversity of materials which they cover; the comprehensive character of the individual specifications wherever inclusion of several grades, sizes, tempers, etc., of one broad classification of material may be advantageously accomplished; and, the generally high quality and close tolerance material which they define. Development in the supply industries, along with rapid improvements and innovations in our own products, result in constant pressure toward reissuing specifications and toward writing new ones covering recently developed materials.

The large number and variety of materials which we use results in a considerable burden in writing and maintaining these specifications, but we consider the returns to be far in excess of the costs involved. Since we have been unable to cover all materials, we still find it necessary to do considerable buying by the less desirable methods of using trade names or individual ordering descriptions. We therefore feel that we are in a position to reach a fairly good conclusion as to the relative values of these methods of buying.

In considering the advantages and disadvantages of buying on a specification basis, we must keep in mind the character of the specification—whether comprehensive or specific, narrow or broad in limitations, whether the material is commercial or non-commercial, new or old—whether the advantages or disadvantages of using the specification are short term or long run as affecting costs and savings, and the volume of purchases to be made under the specification.

The general character of material specifications is governed largely by the class of materials described. Those on products whose requirements are mainly chemical are normally specific and are ordinarily adapted to exact quantitative tests. On items where such properties as color must be defined, there is of course, a problem of a different character, as these are not as readily adapted to commercial testing. While there are available various forms of apparatus for accurate determinations of color, we are for the present using visual comparison of standard samples with the purchased material as a basis of inspection, because of the elaborate nature of other types of tests.

The comprehensive type of specification is of course very useful to large companies in reducing clerical expense. However, there are certain disadvantages in this connection, for, while specifications on such materials as ferrous and non-ferrous sheet, strip, rod and shapes can be made very comprehensive, the difficulty of adequately covering properties for all the contingencies of use of the material is multiplied as the scope of the specification is widened if at the same time it is not made correspondingly voluminous.

The problem of writing and using specifications on a commercial basis may assume very different aspects depending on whether the material to be described is relatively old or whether the result of a new and perhaps incomplete development in the supply industry. The research and development work carried on by the larger industries keeps them on the frontier of technical progress and results in the demand for adaptation of new materials to their needs while such products are still in the producer's development or early commercial stage. Under such conditions, the possibilities of using specifications to good advantage may be small. On the other hand the technical advances in some of the older supply industries have been in some cases so limited as to make writing specifications for their products almost as difficult as for products of the most recent research efforts. Naval stores and sheet ferrous products provide examples of the older materials which present knotty problems in handling on a commercial basis with complete and adequate specifications. Regenerated cellulose and the hard alloys, tantalum carbide and tungsten carbide, are examples of materials which are so recent in development and so much under the control of relatively few companies that the specification of their properties is of doubtful value at present.

If we set up an itemized list of the advantages of specifications to the producer, consumer and buyer, we find that the majority of items are common to the producer and consumer—and to the seller and buyer when identified respectively with the producer and consumer. There are advantages to the purchasing organization as distinct from the consumer. The advantages of specifications to the three parties as we see them are as follows:

First, to the Producer or Seller they:

1. Tend to provide open markets through non-recognition of trade names or fancied superiority of branded products.

2. Stabilize the producer's processes and enable him to set up standard practices of manufacture.

3. Reduce expense caused by trial and error method of setting up to make material suitable for a specific part or use.

4. Minimize delays, misunderstandings, correspondence and discussion resulting from inaccurate or incomplete ordering information.

5. Provide uniformity of testing methods with consequent reduction in inspection discrepancies and costs of handling rejected material.

6. Promote better understanding by consumer of producer's problems through discussion during the process of setting up specification requirements.

7. Tend to reduce production costs through standardization of grades of materials.

8. Tend to promote profitable research by producers as improved technique is often necessary to meet difficult specification requirements.

To the Consumer or Buyer they:

1. Tend to increase the number of sources of supply and aid to reduce inventories through increasing the availability of material.

2. Aid to insure positive control of materials, and, therefore, reduce process difficulties and detect process troubles through the elimination of the material factor.

3. Minimize necessity for shop trials.

4. Reduce production delays incident to receipt of unsuitable material.

5. Provide a workable basis of acceptance and rejection with consequent reduction in controversies with supplier over rejected material.

6. Indirectly increase knowledge of properties, possibilities and applications of materials through the study of requirements in preparing specifications.

7. Tend to promote standardization, from which the consumer finally benefits in lower prices.

8. Decrease handling costs at consumer's plant when efficient packing and shipping methods are specified.

9. Aid designers of the product by providing material information.

To the Purchasing Organization they:

1. Provide information as to the exact type and quality of material required, and, therefore, provide a relatively permanent and standard basis for bids.

2. Give opportunity to make savings through unhindered choice of materials according to price and other purchase factors.
3. Tend to remove trade name restrictions thereby increasing sources of supply and availability of material.
4. Allow more efficient separation of the engineering and purchasing considerations.
5. Reduce volume of correspondence and discussion on individual orders.

I believe that the disadvantages claimed by the opponents of specifications are often based on experience with poorly written specifications. The undesirable effects of such specifications on the producer, consumer, and buyer, alike, are obvious. Some of the faults of ill-considered specifications are:

1. Unnecessary limitations due to ignorance of the properties governing successful and economic use of the material.
2. Ambiguity, resulting in continued controversy.
3. Being unnecessarily out of line with standard or commercial practices.
4. Inclusion of limitations which the supplier cannot reasonably be expected to meet since the origin of the variation is beyond his control.

Such faults produce trouble for all concerned and inevitably result in increased costs for the purchaser.

In listing these advantages of a wider specification use, we have in mind, as a basis of comparison, the methods of buying by trade name or by general description. The use of trade names in purchasing is a time-honored custom and has some real advantages. The tool steel industry furnishes a conspicuous example of merit in this direction, but paradoxically also furnishes the worst example of the degree of confusion and duplication which a multitude of trade names can cause. The tool steel maker has protected his reputation since the early development of the industry by using brand names, and it appears that a real necessity for such procedure has existed. The importance of individual knowledge of the tool steel craft has until recently practically prevented development of specifications which have been of any marked value in differentiating between good and bad tool steels by routine inspection methods. It seems to us, however, that even incomplete tool steel specifications may be of some help in straightening out the situation with reference to the multiplicity of brand names and in making some attempt at standardization. In the matter of tests, the tool steel makers themselves are now doing valuable work which may result in development of specification requirements.

Another field for trade names is of course that of materials manufactured under closely held patents. As a classic instance, "Celluloid" long held the field until other developments on the same general type of material permitted its general specification as transparent or opaque nitrocellulose sheet. "Cellophane" holds today a position somewhat similar to that formerly enjoyed by celluloid.

The name "duralumin" has become a common designation for the well-known alloy of aluminum and copper, and is now generally applied to the metal in much the same way as the name "celluloid" has been applied. Specifications on such materials as duralumin and other aluminum alloys, the production of which may be practically controlled by a single company, are still advantageous from the standpoint of control of the uniformity of that supplier's product, although perhaps not having the other valuable feature of providing for purchase from any acceptable source of supply.

As has been stated, the majority of difficulties and disadvantages arising from the use of specifications are caused by ill-advised or unnecessary limitations. These difficulties are usually aggravated by the fact that the supplier recognizes their lack of justification. It may be well, therefore, to discuss briefly some of the methods by which suitable specification limits can ordinarily be selected.

The Western Electric Company follows the procedure of submitting its proposed specifications to prospective suppliers for criticism, and comments or approval. The supplier's technical staff is urged to discuss the proposed requirements with us in considerable detail, and careful consideration is given to each criticism or suggestion. If advisable, the specification is then revised to conform with the comments received and the suppliers again requested to accept it. After official acceptance, we feel justified in insisting that the specification be met in every respect, and in rejecting material not conforming to it. This policy is, of course, tempered by mitigating circumstances, but we feel that specifications are worthless unless their requirements are insisted on and backed up by a fair policy of justifiable rejection.

Selection of the limits is generally based upon two sets of considerations—those affecting the consumer's process and those affecting the functions and desired length of service of his finished products. As a rule, we find that process considerations form in general a simpler problem in definitions and measuring sticks than do the problems of functioning and life in service. The properties governing service life are usually determined by a research organization and form the basis for the original selection of the material to be used for a given product. The reason for this is that tests for duration of service under varying conditions of fatigue, corrosion, stress, wear, exposure to light and heat, etc., are fundamental studies and best suited to the research laboratory type of investigation. The Western Electric Company is furnished with many material requirements based on tests of this nature by the Bell Telephone Laboratories, its associated design and research organization. The results of fundamental studies made by the Laboratories appear in engineering specifications for the material as requirements affecting its composition, temper, strength or hardness, and are translated into commercial terms in the purchasing specification, along with considerations arising purely from manufacturing problems. Where materials are used only for minor structural purposes and play no definite part in the functioning of the product, the process considerations are governing. If, however, stress, fatigue, or corrosion resistance are of major importance, the more fundamental design aspects govern, and the process must be adjusted to produce satisfactory parts from a material which meets the major service requirements.

Although more methods of test are continually being devised, the engineer writing material specifications is still seriously handicapped by a lack of a sufficient number of cheap measuring sticks. It is often difficult for him to define materials in such manner as to preclude the possibility that material meeting the specification will not be satisfactory in process. For instance, as yet we know of no suitable inspection tests for iron welding rod; that is, we cannot yet control the welding qualities of this rod by physical, chemical, or other cheap available tests. In such cases we are forced to fall back on actual shop trial.

In many instances lack of sufficient fundamental knowledge on the part of the user works a hardship on the producer in that specifications are closer than might actually be necessary were the effect of variations accurately known. A well-known instance of this is the large factor of safety which has in the past been applied to the percentage of phosphorus and sulfur allowed in steel, although in recent years it has been demonstrated that these elements have been unjustifiably feared as applied in reasonable proportions to most uses of steel. It is increasingly becoming recognized that the ends of specification writing are defeated if the requirements are not based on production or economic necessity. The limits which have been placed on the area per pound or thickness range of our condenser aluminum foil furnish an illustration of the desirable type of selection. These limits proceed logically from the necessities of manufacturing practice and the final requirements of the product. To wind paper condensers small enough to meet space limitations in telephone exchanges, it has been determined by actual trials that a certain minimum area per pound of foil is necessary. It has likewise been determined by actual trials that the maximum area per pound is very definitely limited by breakage of excessively thin foil during the winding operation.

If a specification is properly written, its use results in reduction of trial and error in the producer's plant and aids him in establishing a manufacturing practice, as well as guiding him in his selection of raw material.

An advantage to the producer, which is sometimes overlooked but which has on several occasions been of marked importance to him in connection with Western Electric specifications, is the advancement of his own research program through the necessity of meeting requirements which, at the time originally put up to him, were beyond the limitations of his manufacturing technique. Although such research may cause the supplier initial expense, later returns on this investment may be very profitable. For instance, the development work done by domestic paper manufacturers in order to supply us with condenser paper meeting our exacting requirements, resulted in a good return to these manufacturers when the demand for radio condenser paper developed. These manufacturers were then in a position to supply paper which had formerly been produced only in Europe. Another example of this type is the impetus which was given by our company to the development in this country of high-speed screw machine steels. As the result of our negotiations with a domestic supplier, a steel was produced comparable to the best then available from abroad, and since that time, practically every important manufacturer of cold-drawn screw machine steel has been keenly interested in following up this development to the ultimate degree of fastest machining properties consistent with sufficient physical strength.

There is at least one other advantage from the purchasing viewpoint which is not generally considered. Specifications provide a basis not only for obtaining but also in some instances, for analyzing competitive bids. Merely to compare the bids received from several suppliers at any one time is not sufficient for the purpose of—what technical men will identify as—scientific purchasing. For in addition to selecting the lowest bidders, we want to know how the prices offered at present compare with the past and how these quotations are related to the cost of producing the material purchased. When the purchased product is of relatively simple fabrication so that the contained raw materials constitute the principal element of cost, it is a simple matter to answer these questions—with the aid of specifications. For example in buying products like brass and bronze, composed of several metals whose market prices fluctuate frequently and considerably, changes in price of the finished products are to be expected. Specifications, which of course indicate the proportion of each metal contained in the finished products, give a direct clue to the cost structure. By weighting the current prices of the individual metals by the quantity of each contained in any item, we derive the value of the total metal content. The difference between this amount and a supplier's quotation represents that supplier's manufacturing differential for the particular item. We can then compare these manufacturing differentials of several suppliers for a single item, or of one supplier for different items whose method of manufacture is similar, or for the same item at different times and different raw material prices. Such a study of prices reveals the effect of improved methods of manufacture and the state of competition within an industry. . This method of attack is adaptable to such diversified products as insulated wire—with its copper, rubber, cotton and waxes; to dry cell batteries—with their zinc, manganese, etc.; to corrugated paper cartons with their jute, Kraft, chip or straw board liners; even to liquid soap, with its potash and cocoanut oil. These and many other materials can be so analyzed because an adequate material specification serves as the X-ray to penetrate beneath the surface of trade custom, brand markings and sales talk.

As has been stated, the chief purposes of specifications are to define material accurately and to provide tests which will insure that material meets the limitations of the definitions. Certainly the methods of test constitute an essential part of a purchasing specification and it is, of course, along these lines, that the American Society for Testing Materials has done so much important work. The Western Electric Company has made extensive use of A.S.T.M. methods of test in raw material specifications, some of which refer to several A.S.T.M. methods. Also, we have cooperated in the development of a number of the A.S.T.M. specifications and methods of test. Perhaps the most important of the developments in which we have cooperated have been the "J" or Babbitt permeameter for magnetic measurements, physical requirements and testing standards for non-ferrous sheet metals, and the work on the composition limits of die-casting alloys and the properties of condenser papers.

The trend in specification work continues to be toward simplification and standardization, more cooperative work between producers, buyers, consumers and technical societies, and added use of information made available by the good work of societies such as the American Society for Testing Materials, the American Society for Steel Treating, and Society of Automotive Engineers.

# DISCUSSION ON MOTOR OILS

MR. H. C. MOUGEY[1] (*presented in written form*).—The problem of specifications for motor oils is different from most specification problems in that the volume of sales is by retail rather than by wholesale. Steel and cement are purchased by a few large companies but the general public buys most of the motor oil sold and these purchases are usually in very small units. Steel and cement can be inspected and tested by the large companies that buy them, but motor oil is bought by the public on faith. Since motor oils are made with widely differing properties and since these properties greatly affect the performance in service, it is evident that specifications for motor oils are of great importance to the automobile industry and the oil industry as well as to the general public.

One of the most important uses of specifications is for purposes of classification and recommendation of the types of lubricants that will give best results under the various service conditions. The Society of Automotive Engineers recognized this problem several years ago and, after trying in vain to solve it for several years, their Lubricants Committee appealed to the American Society for Testing Materials for help. As a result of this cooperation, the S.A.E. crankcase oil numbers were adopted by the S.A.E. and they are now in general use. These numbers make it possible for the automobile companies to recommend the proper lubricants for different conditions, the oil companies can supply the lubricants recommended, and the public can obtain lubricants of the desired kinds.

The S.A.E. numbers are limited to viscosity requirements and the oil companies are responsible for the other factors of quality. Quality in motor oil is very difficult to define. Perhaps this is because quality is not inherent in the oil itself but depends upon its being suited to the engine and the conditions of service. A winter oil of high quality might be very bad for summer, and *vice versa*. Many other examples might be given.

The American Society for Testing Materials has recognized this difficulty and its Committee D-2 on Petroleum Products and Lubricants has organized a committee, designated Technical Committee B, to study this problem. At the present time, most oil specifications which go into any detail at all simply specify certain tests and limits which experience has shown will fit a certain oil. Other oils, perhaps of equal quality, would require other limits. In other words, they are identification tests, but whether or not the oil meeting the tests and limits is desirable is another question.

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Technical Committee B which is studying this problem decided that, instead of specifying some identification tests and then assuming that oils which meet these tests would be satisfactory in service, the problem should be attacked from the angle of performance in service. The service requirements that the oil must meet were studied with the view of specifying these requirements rather than the identification tests of the oils. The work has really just started but the results are already becoming apparent. Attention has been focused on certain weaknesses of some oils and to certain fundamental weaknesses in design in automobiles. Both the oil industry and the automobile industry have started work trying to correct these weaknesses, and the general public is getting better oils, better engines, and more satisfactory service from both.

This example illustrates one of the advantages to be derived from specifications. They are very valuable as tools for research and development. When we try to write down definitely what properties and requirements are *necessary* and *why*, it frequently happens that we open up the way to get around these difficulties. The oil industry has been doing a great amount of work in trying to decrease the rate of change of viscosity with change of temperature in motor oils. The automobile industry is working on this same problem from the angle of decreasing the total temperature change so that the resulting viscosity change will be less. The work of both industries results in having oils which do not change so much in viscosity in service.

The oil industry has been devoting much time and money to decreasing the pour tests and carbon tests on their oils. Some of the automobile companies are beginning to work on making their engines better suited to the lower viscosity oils which are fundamentally lower in carbon test and which can be made low in pour test more easily.

From this brief discussion it is seen that oil specifications are of three kinds:

1. The S.A.E. viscosity numbers which constitute a classification for purposes of recommendation. This type of specification is the one which experience has shown to be most desirable for use by the general public.

2. Identification tests, sometimes called purchasing agents' specifications. These are identification tests which make it possible for a

certain oil to be ordered and its identity checked when it is received. There is no assurance that many other oils which might not meet the tests and limits might not be much better for the desired service. This type of specification is very valuable, but unless it is properly used, it tends toward stagnation and the blocking of progress for both the oil industry and the automobile industry.

3. Quality specifications. These are only beginning to come into use. They depend upon the proper coordination of oil and the use to be made of it. This is the problem upon which Technical Committee B is now working. Work on quality specifications is of great value in removing unnecessary restrictions and in developing better oils and better engines.

As a result of the activities of the national technical societies, the oil industry and the automobile industry are cooperating in a very wonderful manner. Much progress has been made, but there is more to be done, and it is only by the continued cooperation of these two industries that we can hope to solve our mutual problems.

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